DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02254

REPLY TO
ATTENTION OF:

NEDED-F

18 December 1981

SUBJECT: Reconnaissance Report, Surry Mountain Dam, Keene, NH

CDR (DAEN-CWO-M)
WASH, DC 20314

In accordance with ER 1110-2-417, there is submitted for review and approval Reconnaissance Report, Surry Mountain Dam, located in the Connecticut River Basin, Ashuelot River, Keene, New Hampshire. As recommended in paragraph 6, a reply by 20 January 1982 would be appreciated to allow for subsequent activities to proceed as scheduled.

FOR THE COMMANDER:

B Beard

JOE B. FRYAR, P. E.
Chief, Engineering Division

Inclosure (10 cys)
As stated

EARTHQUAKE DESIGN AND ANALYSIS FOR CORPS OF ENGINEERS DAMS

RECONNAISSANCE REPORT

FOR A

SPECIAL ENGINEERING INVESTIGATION

SURRY MOUNTAIN DAM

KEENE, NH

US Army Corps of Engineers
New England Division
Engineering Division
Geotechnical Engineering Branch
Waltham, Massachusetts 02254

December 1981

5

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EXHIBITS

1. AUTHORITY. The authority for this study and report is contained in the following:

ER 1110-2-1806, Earthquake Design and Analysis for Corps of Engineers Dams, 30 April 1977.

ER 1130-2-417, Major Rehabilitation and Dam Safety Assurance Program, 30 November 1980.

EC 1110-2-229, Special Engineering Investigations, Dam Safety Assurance Program, 18 March 1981.

2. PURPOSE. Earthquake analyses performed to date indicate a potential safety problem at Surry Mountain Dam under earthquake induced loading. The purpose of this report is to justify the need for a special engineering investigation.

3. CURRENT STATUS. The current status of the New England Division's seismic evaluations and analyses program was presented in letter to the Commander, U. S. Army Corps of Engineers, DAEN-CWE-SS, dated 14 July 1981 in response to a letter request from the Chief, Engineering Division, Director of Civil Works dated 11 June 1981 (Exhibit A). The reply letter affirmed that thirty-five completed New England Division dams which were analysed by the pseudo-static method in accordance with the criteria in ER 1110-2-1806, have adequate factor of safety. Six of the thirty-five dams were also analysed for liquefaction and cyclic mobility potential and three out of six were found to have a potential seismic instability problem; these dams are: Knightville, Surry Mountain and West Thompson. The dynamic stability analysis of Knightville Dam is in progress and scheduled for completion in October 1982; Surry Mountain and West Thompson are scheduled for investigation in FY-82 and 83 respectively.

4. EMBANKMENT AND FOUNDATION

a. General. Surry Mountain Dam was constructed in 1939-41 and designed by the U. S. Engineer Office, Providence, RI; Re: "U. S. Army Corps of Engineers Design Memorandum, Connecticut River Flood Control, Surry Mountain Dam, Ashuelot River, NH: Analysis of Design", July 1939. A summary of embankment and foundation conditions extracted from the Analysis of Design is in Exhibit B.

b. Dam Embankment. The dam embankment is a rolled-earth-fill 86 feet maximum height approximately 1,800 feet long, consisting of a central impervious core flanked by random impervious and pervious shells. (Exhibit B).

c. Foundation Conditions. The foundation is composed of three types of glacial sediments; stratified outwash sand and gravel, glacial till and glacial lake deposits of uniform fine sand and silt (Exhibit B), overlaying rock which is about 100 foot depth at the center of the valley and left abutment and rock outcropping on the right abutment. Deposits of loose, low strength, finely textured fine sand and silt occupy a prominent portion of the dam foundation.

On account of the low strength and the possibility of plastic flow or large deformation of the foundation during embankment construction it was decided during design to construct the embankment over a two construction season period to allow weaker soils to gain strength through consolidation. During the first season, the cut off trench was constructed and the site was prepared for embankment construction; the embankment was constructed during the following two construction seasons. Two borings at the landside toe of the embankment made during the 1980-81 investigation (Exhibit C) disclosed low penetration (SPT) resistance values in the foundation fine sand and silt deposit.

5. PRELIMINARY SEISMIC ANALYSES

a. General. Seismic analyses of Surry Mountain Dam were made under the New England Division program of seismic evaluation of its existing dams. The following investigations have been completed to date and copies of the reports have been furnished to Headquarter, Department of the Army (DAEN-CWE-SS).

(1) Remote Sensing Analysis of Fault-Related Structures in New England and Related Seismic Hazards at Corps of Engineers Projects, October 1978.

(2) Low-Sun Angle Aerial Reconnaissance of Faults and Lineaments of Southern New England, September 1980.

(3) Stability Analysis by the Seismic Coefficient Method, Completed New England Division Dams, August 1980.

(4) Liquefaction and Cyclic Mobility Potential, Completed New England Division Dams, Phase I Investigation, September 1980; Phase II Investigation February 1981.

b. Pseudo-static Analysis. A pseudo-static earthquake stability analysis was performed for the steady seepage condition with reservoir pool elevations at the critical pool level and at the maximum pool level for the upstream and downstream dam embankment slopes respectively (see Exhibit D). The minimum computed factors of safety are higher than the required minimum of 1.00 and are as follows:

Condition	Seismic Coefficient	Downstream Slope Factor of Safety	Upstream Slope Factor of Safety
Static	-	1.86	2.03
Pseudo-static	0.05	1.50	1.61
"	0.10	1.26	1.32
"	0.13	1.15	1.19

The seismic coefficient of 0.05 was selected for stability analysis from the Seismic Risk Map in ER 1110-2-1806, the 0.10 coefficient is the next higher value selected from the risk map as directed by the OCE for use in the seismic stability analysis. The 0.13 coefficient used in the stability analyses was derived by using a predicted peak acceleration determined by increasing the recorded Mercalli intensity by one unit; the value thus obtained was attenuated to the site.

c. Liquefaction and Cyclic Mobility Potential. The investigation completed in 1981 (Exhibit B) included two foundation borings, laboratory testing of undisturbed samples and a study on the potential for cyclic mobility and liquefaction. The borings disclosed penetration resistances (SPT) values in the range of 2 to 10 blows per foot (pulley and cathead with a sleeve type hammer) in foundation soil zones of fine sand and silt. These low SPT values indicate a possible potential for liquefaction (see page 4). Cyclic and monotonic triaxial test results performed on undisturbed soil samples indicated that contractive volume changes may occur in the loose fine sand and silt zones of the dam foundation with a potential of liquefaction and cyclic mobility. Page 5 shows a typical curve for contractive and dilative soil. Results of the investigation led to a recommendation for execution of a dynamic analysis of the dam (Exhibit B).

6. RECOMMENDATION FOR A SPECIAL ENGINEERING INVESTIGATION

The investigations performed to date have disclosed that there may be a dam safety problem at Surry Mountain Dam because of the potential for liquefaction and cyclic mobility of its foundation. A special engineering investigation is needed to identify the extent and severity of the problem and the need for remedial construction work. The special engineering investigation will consist of borings, field seismic work, seismicity investigation, laboratory testing and response analysis. Funding for the investigation has been included in the New England Division O&M Budget for FY 82 and a cost estimate breakdown is as follows:

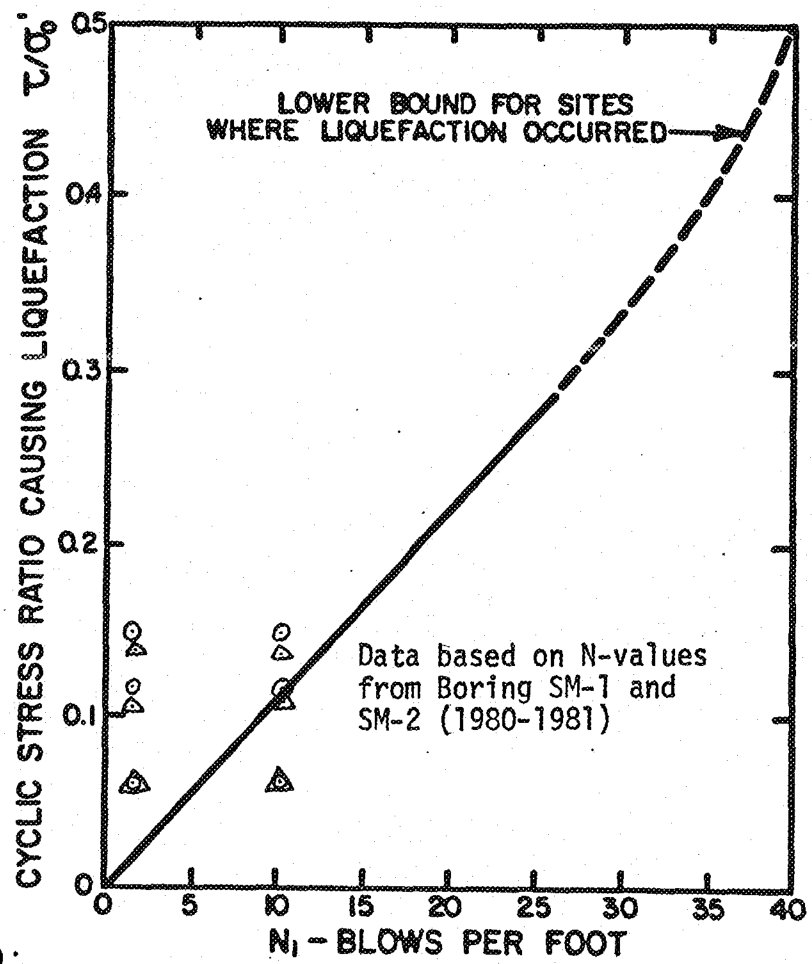
a. Work by Contract (Scheduled for Award in April 1982)

Field Borings and Seismic	$ 70,000	
Laboratory Testing	80,000	200k
Geology and Seismicity	50,000	
Response Analysis	50,000	
Reports and other costs	30,000	
Total est. A-E cost	$280,000	

b. In-House Cost

Recon. Report and Contract Specs.	20,000
Contract management and review work	45,000
Total est. in-house cost	54,000

c. Total Cost $345,000



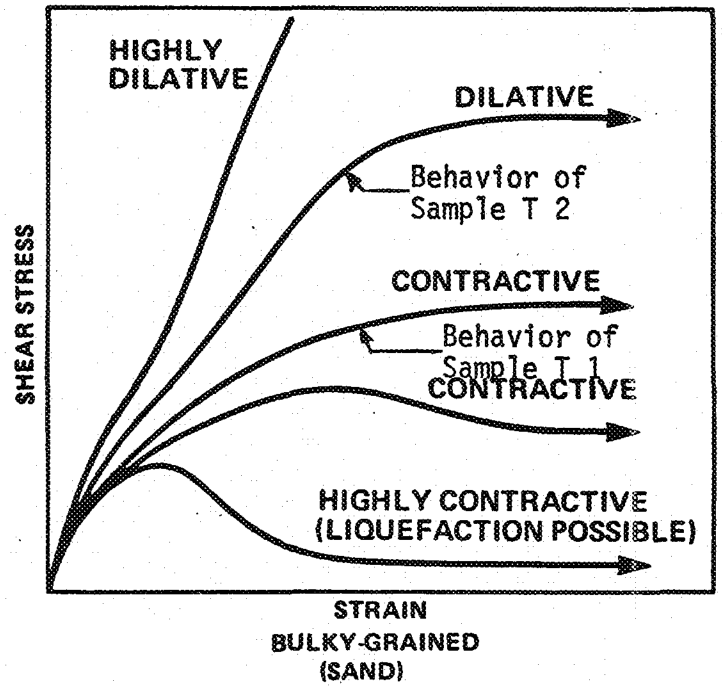
LEGEND:

⊙ Liquefaction - stress ratio based on R_D=0.8 and acceleration of 0.13g, 0.10g and 0.05g at 27 feet below ground surface.

△ Liquefaction - stress ratio based on R_D=0.8 and acceleration of 0.13g, 0.10g and 0.05g at 36 feet below ground surface.

CORRELATION BETWEEN STRESS RATIO ASSUMING TO CAUSE LIQUEFATION IN THE FIELD AND PENETRATION OF SAND (SEED,1976)

EFFECT OF INITIAL VOID RATIO ON STRESS-STRAIN CURVE
UNDRAINED LOADING



GEI-077-575

Chart taken from G E I "Analysis of embankment and soil foundations for earthquake loading" March 14-15,1978.

EXHIBIT A

EXHIBIT A

STATUS OF SEISMIC EVALUATIONS AND ANALYSES OF EXISTING DAMS

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DEPARTMENT OF THE ARMY
OFFICE OF THE CHIEF OF ENGINEERS
WASHINGTON, D.C. 20314

REPLY TO
ATTENTION OF:

DAEN-CWE-SS 11 June 1981

SUBJECT: Status of Seismic Evaluations and Analyses of Existing Dams

SEE DISTRIBUTION

1. Reference ER 1110-2-1806, 30 April 1977, Earthquake Design and Analysis for Corps of Engineers Dams.

2. The referenced regulation is currently being revised and the status of seismic evaluations is needed to accomplish the revision. Each division should review the status of the investigations and evaluations being performed and submit a revised schedule for completion of the program. This revised schedule should include all projects studied, completed and planned by name and purpose, type of structure, date the investigation was or will be started, date completed or planned completion date, the magnitude of the design earthquake at the site, and actual or planned cost of the investigation and/or evaluation. Projects that have been excluded from this program should be listed separately.

3. In cases where ongoing seismic evaluations will lead to future comprehensive studies and/or remedial measures, the report should be submitted to this office for approval.

4. The schedule should be submitted to DAEN-CWE-S by 15 July 1981. The funding summary should be furnished not later than 15 September 1981.

FOR THE CHIEF OF ENGINEERS:

Lloyd A Duscha

LLOYD A. DUSCHA, P.E.
Chief, Engineering Division
Directorate of Civil Works

DISTRIBUTION
See Page 2

DAEN-CWE-SS 11 June 1981

SUBJECT: Status of Seismic Evaluations and Analyses of Existing Dams

DISTRIBUTION:

Division Engineer, Lower Mississippi Valley (ATTN: LMVED-G)
Division Engineer, Missouri River (ATTN: MRDED-G)
Division Engineer, New England (ATTN: NEDED-F)
Division Engineer, North Atlantic (ATTN: NADEN-TF
Division Engineer, North Central (ATTN: NCDED-F)
Division Engineer, North Pacific (ATTN: NPDEN-GS)
Division Engineer, Ohio River (ATTN: ORDED-G)
Division Engineer, Pacific Ocean (ATTN: PODED-G)
Division Engineer, South Atlantic (ATTN: SADEN-F)
Division Engineer, South Pacific (ATTN: SPDED-G)
Division Engineer, Southwestern (ATTN: SWDED-F)

NEDED-F 14 July 1981

SUBJECT: Status of Seismic Evaluations and Analyses of Existing Dams

CDR USACE (DAEN-CWE-SS)
WASH DC 20314

1. Reference your letter of 11 June 1981, subject as above (copy attached) and to telephone discussions of 7 and 8 July 1981 with Mr. A. Walz.

2. Starting in September 1977, NED has been executing a program of seismic evaluations of its existing dams. Of the thirty-nine existing NED dams, thirty-five have been considered, the other four dams were excluded because their failures would not endanger lives or vital installations. The attached Table I is a listing of the dams covered by this program which includes purpose, type and peak accelerations for the design earthquakes for each site and a listing of the excluded dams.

3. Regional and detailed tectonic investigations have been completed for all thirty-five dams. The peak accelerations shown on Table I are based on the historical earthquake activity. All thirty-five dams have been analyzed for seismic stability by the pseudo-static method. Results of the analyses show factors of safety of unity, or greater, for the seismic coefficients for the seismic risk zones in which the dams are located and for the next higher zones. Reports covering these investigations and analyses were forwarded on 14 January 1981.

4. Six dams were investigated for their potential for liquefaction and cyclic mobility. The Phase I and Phase II reports for this investigation were forwarded on 14 January 1981 and 19 March 1981 respectively. Based on the results of this investigation, three dams have been selected for dynamic analysis for stability and deformation under earthquake loading. The dynamic analysis investigation is in progress for one dam, Knightville, and is planned for Surry Mountain and West Thompson starting in FY-82 and 83 respectively.

5. The attached Table II presents the status of NED activities for this program including start and completion dates and a funding summary.

FOR THE COMMANDER:

3 Inclosures
as

JOE B. FRYAR, P. E.
Chief, Engineering Division

CF
Proj. Mgt. Br. (Mr. Gould)
Operations Div. (Mr. Minior)
GEB Files
Eng Div Files (1125)

TABLE I

EXISTING NED DAMS INVESTIGATED UNDER ER 1110-2-1806

(EARTHQUAKE DESIGN AND ANALYSIS FOR CORPS OF ENGINEERS DAMS)

JULY 1981

	DAM	PURPOSE	TYPE	PEAK ACCELERATION*(g)
1.	Ball Mountain	FC	E-R	0.18
2.	Barre Falls	FC	E-R	0.18
3.	Birch Hill	FC	E	0.18
4.	Black Rock	FC	E	0.25
5.	Blackwater	FC	E-R	0.18
6.	Buffumville	FC	E	0.18
7.	Colebrook River	FC-WS-R	E-R	0.25
8.	Conant Brook	FC	E-R	0.18
9.	East Branch	FC	E	0.18
10.	East Brimfield	FC-R	E	0.25
11.	Edward MacDowell	FC	E-R	0.18
12.	Everett	FC-R	E-R	0.18
13.	Franklin Falls	FC	E-R	0.25
14.	Hall Meadow Brook	FC	E	0.18
15.	Hancock Brook	FC	E	0.25
16.	Hodges Village	FC-R	E-R	0.18
17.	Hop Brook	FC-R	E	0.25
18.	Hopkinton	FC-R	E-R	0.18
19.	Knightville	FC	E	0.18
20.	Littleville	FC-WS	E-R	0.18
21.	Mad River	FC	E	0.18
22.	Mansfield Hollow	FC	E	0.25
23.	Northfield Brook	FC-R	E-R	0.25

	DAM	PURPOSE	TYPE	PEAK ACCELERATION*(g)
24.	North Hartland	FC-R	E	0.18
25.	North Springfield	FC-R	E-R	0.18
26.	Otter Brook	FC-R	E	0.18
27.	Sucker Brook	FC	E-R	0.18
28.	Surry Mountain	FC-R	E-R	0.13
29.	Thomaston	FC	E-R	0.13
30.	Townshend	FC-R	E	0.18
31.	Tully Lake	FC	E-R	0.18
32.	Union Village	FC	E-R	0.18
33.	West Hill	FC-R	E	0.18
34.	West Thompson	FC-R	E	0.18
35.	Westville	FC-R	E	0.18

*Peak acceleration at site for design earthquake

LEGEND

FC - Flood Control
WS - Water Supply
R - Recreation
E - Earth Fill
R - Rock Fill

Existing NED Dams not included in program because failure would not endanger lives or vital installations.

a. Charles River - Run-of-river dam to control fresh water level in tidal basin.

b. Cherryfield - Log-crib run-of-river dam for ice jam control.

c. Wright Reservoir - Low level dam (17-foot) for urban drainage system.

d. Smelt Brook - Low level dam (15-foot) for urban drainage system.

TABLE II
STATUS OF SEISMIC EVALUATIONS AND ANALYSES OF NED DAMS
JULY 1981

Activity	Dams Covered	Dates Start	Dates Complete	Funding Status (Thousand Dollars)	Report Titles
1. Regional Tectonic Investigation	35 dams (See Table I)	Sep 77	Oct 78	19.2	*Remote Sensing Analysis of Fault Related Structures in New England and Related Seismic Hazards at Corps of Engineers Projects
2. Detailed Tectonic Investigation	35 dams (See Table I)	Jan 78	Sep 80	32.8	*Low-Sun Angle Aerial Reconnaissance of Faults and Lineaments of Southern New England
3. Pseudo-static Stability Analysis	35 dams (See Table I)	Jan 80	Oct 80	77.5	*Stability Analyses by the Seismic Coefficient Method-Completed New England Div. Dams
4. Liquefaction and Cyclic Mobility Investigation	Franklin Falls Surry Mountain Knightville Hodges Village West Thompson Mansfield Hollow	Jan 80	Jan 81	180.5	*a. Liquefaction and Cyclic Mobility Potential - COE Completed New England Dams - Phase I Investigations **b. " Phase II Investigations
5. Dynamic Analyses	a. Knightville	Mar 81	Dec 82(est)	36.6(exp) 377.4 (budgeted)	
	b. Surry Mountain	FY-82	FY-83	344.0 (budgeted)	
	c. West Thompson	FY-83	FY-84	360.0 (budgeted)	

Total Expended	346.6
Budgeted	1,081.4
Grand Total	1,428.0

Copies furnished DAEN-CWE-SS by ltr of 14 Jan 81
* Copy furnished DAEN-CWE-SS by ltr of 19 Mar 81

EXHIBIT B

EXHIBIT B

SUMMARY OF EMBANKMENT AND FOUNDATION CONDITIONS*

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*The material in this exhibit was obtained from the report entitled "Liquefaction and Cyclic Mobility Potential, Corps of Engineers Completed New England Dams, Phase I and II Investigation, Sept 1980 and Feb 1981 respectively.

SURRY MOUNTAIN DAM

1. GENERAL

The Surry Mountain Dam is located on the Ashuelot River, a tributary of the Connecticut River, about five miles northwest of Keene, New Hampshire (see Page B-6). The dam is one of a series included in the "Revised 1936 Flood Control Act Project for the Connecticut River Valley," and was constructed in 1939 and 1941.

The dam is a rolled-earth fill structure having a maximum height of 86 feet and a total length of approximately 1,800 feet. As shown on Page B-7 the embankment consists of a "select impervious"core that is tied to the foundation with a cutoff trench, and "random impervious" and "pervious" shells. Upstream and downstream slopes are riprapped with dumped rock, and vary in slope from 2.5 horizontal (H) to 1 vertical (V) between Elevation 565 feet and Elevation 550 feet, to 3H to 1V between Elevation 550 feet and Elevation 520 feet, to 5H to 1V below Elevation 520 feet. Rock toes exist on each slope. The outlet works and the main spillway weir are constructed on rock in the right (west) abutment.

2. GEOLOGY

The Ashuelot River is situated in a rugged upland area composed of igneous and closely folded metamorphic rocks, which at some locations occur at the surface due to weathering and pre-glacial stream erosion and at other locations are covered by glacial deposits. Igneous rock or granite is exposed near the right abutment of the dam, but as much as 100 feet of glacial overburden remains in the middle of the valley and near the left abutment.

Three types of glacial sediments comprise the overburden: (1) stratified outwash sand and gravel, (2) unstratified glacial till, and (3) uniform glacial lake deposits. These deposits are described in the next section in more detail, especially the glacial lake deposits which are felt to be the least resistant to liquefaction and cyclic mobility. The interstratified deposits of finely textured sand and silt are predominant in the foundation of the dam.

The lower part of the valley consists primarily of deposits of outwash sand and gravel. These deposits occur in partially eroded terrace formations, and are coarsely granular and pervious. Glacial till occurs in most of the hillside that forms the left abutment. This formation is well graded, devoid of bedding and very compact, making it relatively impervious.

3. FOUNDATION CONDITIONS

Previous Exploration. Prior to construction of the dam, subsurface exploration programs were conducted using core borings, test pits and auger borings. Fifty-eight core borings were advanced to explore a total of approximately 3,200 feet of subsurface material, including more than 300 feet of rock core and more than 340 feet of undisturbed soil samples. A total of twenty-four test pits were dug in the area of the foundation and 124 in borrow areas; 128 auger borings were also used to explore both foundation and borrow areas. Boring locations are shown on Page B-8 and boring logs are shown on Pages B-9 thru B-11. Note that no penetration resistance was measured while advancing spoon samplers, and therefore, no quantitative description of density (such as SPT resistance) is provided on the boring logs.

The exploration program permitted cross-sections of subsoils to be developed as shown on pages B-12 and B-13 . Page B-13 indicates that the foundation soils along the centerline of the dam are primarily interbedded sands and silts that extend to a maximum depth of approximately 100 feet. The Providence District system of soil classification was used. This system, summarized on Page B-13 classifies soils from Class I (clean gravel) to Class 13C(variable clay). It is important to note that soils designated by even numbers are uniformly graded, while those designated by odd numbers are well graded. Accordingly, Class 6 material (uniform fine sand to coarse silt) for example, might be expected to exhibit quite different liquefaction potential or cyclic mobility than Class 7 material (variable graded from gravel to coarse silt).

It may be seen from Page B-12 and B-13 that the shallow overburden in the right abutment consists of uniform fine sand and glacial silt (Classes 6, 8, 10 and 12) overlain by sand and gravel (Classes 2, 4, 5 and 7). The left abutment contains unstratified, dense glacial till consisting primarily of Classes 7, 9 and 11. However, the foundation of the embankment is composed of thick deposits of finely laminated and interstratified sediments designated as Classes 8 and 10, which are overlain and underlain by sands (often Class 6) and gravels.

It may be seen from the laboratory test data furnished on Page B-13 that void ratios for Class 10 material (uniform medium to fine silt) range from approximately 0.63 to 1.04, mean value being approximately 0.85. These void ratios presented for Class 8 material range from 0.61 to 0.76, which are also rather large. Accordingly, Class 10 soils would appear to exist at a relative density not much higher than 40 percent.

The low density of these soils is also indicated by low strengths. The angle of internal friction for the silt varies from approximately 23 to 34 degrees and the cohesion is negligible (less than 10 percent of the soil is clay). Low strength and the possibility of large plastic deformation or flow of the silts within the foundation soils was a concern of the designers with regard to stability, and led to the determination of consolidation characteristics of the material. It was decided to extend the duration of embankment construction to nearly two years to permit the weaker foundation soils to strengthen gradually through consolidation; also, the slopes were flattened to 5 horizontal to 1 vertical in the lower third of the embankment, presumable to provide both additional confining pressure and reduction of shear stress on the foundations.

Recent Borings. Two borings (SM-1 and SM-2) were made at the downstream toe of the dam to investigate the loose foundation soils. These borings were taken thhough the spoil fill beyond the rock toe. Boring SM-1 was taken to refusal at a depth of 83 feet and Boring SM-2 was taken to 80.5 feet. Standard penetration tests (SPT) were taken at 5-foot intervals except in the loose zone where undisturbed samples were taken in SM-1. In SM-1 between 26 and 55 feet, four 3 inch undisturbed Shelby tube samples were taken. In this zone, SPT's were taken at approximately 10 foot intervals.

Borings SM-1 and SM-2 indicate that there are approximately 50 feet of very loose to loose, stratified uniform very fine sand and silt with occasional thin, stiff silty clay layers. The upper portion is somewhat finer than the lower portion of the stratum. Laboratory gradation and water content tests* were performed on selected representative samples in the stratum. Based on water content and specific gravity determinations, the void ratios of the silt and fine sand are between 0.9 and 1.2. The sand below about 50 feet has void ratios in the range of 0.4 to 0.7. SPT values* in the silt and fine sand stratum were generally beween 2 and 10 blows per foot. Below the stratum is about 20 feet of medium dense, stratified fine sand and gravelly sand. SPT values were generally between 15 to 20 blows per foot in this lower stratum.

4. TRIAXIAL TESTS ON FOUNDATION SOILS

Because void ratios were found to be large and SPT resistance low, it was considered advisable to conduct both undrained monotonic and cyclic tests on saturated samples of the foundation soils. The stress-controlled monotonic tests would indicate whether these soils exhibited dilative or contractive behavior when sheared, and hence whether a flow condition could occur in situ. The cyclic triaxial tests would indicate the extent of cyclic mobility that might be anticipated under the imposition of seismically induced shear stresses. Tests conducted on soils at Surry Mountain (three monotonic and two cyclic tests) were intended to be exploratory only. Conclusions based on results of these tests are therefore tentative, and should be based ultimately on more extensive laboratory test data.

Two monotonic stress controlled triaxial tests were conducted on specimens recovered from Sample T2 in Boring SM1 at a depth of approximately 36 feet. The two specimens comprised of fine sand with some silt had initial dry unit weights of 101.7 pcf and 97.8 pcf, respectively. The plots of deviator stress versus axial strain and pore pressure versus strain* showed marked dilation when sheared. Accordingly, the points shown on the plot of void ratio versus effective confining pressure will lie below the flow line (associated with actual liquefaction) for this soil. It should be noted that the densities of the soils tested may not be representative of most of the foundation soils, as the SPT resistance tended to be somewhat higher around the depth of 36 feet compared with that at lesser and greater depths. Moreover, the very loose silt stratum found to exist between depths of 12 feet and 27 feet is expected to contract (and thus flow) when sheared.

*Test results are shown in Exhibit C

A single monotonic triaxial test was conducted on a reconstituted specimen comprising of silt and fine sand at an initial dry density of 92.2 pcf. (The material tested was that used previously in cyclic tests conducted on undisturbed samples, as described below.) As shown on the plots of deviator stress versus strain and pore pressure versus strain*, this specimen exhibited contraction during shearing. Accordingly, it is likely that at least part of the silt stratum may exist at a void ratio greater than the critical value.

Two cyclic triaxial tests were conducted on undisturbed silt samples recovered in Sample T1 from Boring SM-1 at a depth of approximately 27 feet. The initial dry densities of the two specimens were 93.6 pcf and 91.2 pcf respectively. As shown on the plot of shear stress ratio versus number of cycles, the soil exhibited a large accumulation of pore pressure and 5 percent strain (peak-to-peak) in slightly more than one cycle for a stress ratio of 0.3, and 10 percent strain at three cycles. For a stress ratio of 0.2, 5 percent and 10 percent strains occurred at 12 cycles and 20 cycles, respectively. These low cyclic strengths further indicate the contractive nature of the silts encountered in the foundation soils.

5. DAM EMBANKMENT

The rolled-earth fill method, rather than a hydraulic fill method, was used for construction of the embankment, primarily to make a two season construction schedule economical. The foundation was expected to consolidate and strengthen sufficiently during construction to permit adequate stability against plastic flow.

The embankment materials were procured from local borrow areas and were placed by trucks or crawler wagons and compacted by sheepsfoot rollers. The embankment is composed of four types of fill:

(1) pervious fill
(2) impervious and random impervious fill
(3) rock toes and dumped riprap
(4) gravel filters and bedding

According to the design memorandum, the pervious fill was to consist of Classes 2, 3, 4 and 5 soils, and were to be placed in such a manner that the finer materials would be nearer the random impervious section; the coarser materials were to be placed nearer the outer faces of the embankment. The impervious material consists of Classes 7, 9 and 11. All soils were to be either dried or moistened to near optimal water content, spread in 6 inch thick layers, and rolled. Standard Proctor tests yielded moist weights of 120 to 140 pcf at optimal water contents of 9 to 16 percent for pervious materials; moist unit weights of 135-145 pcf were obtained at optimal water contents of 8 to 14 percent for impervious materials. The impervious section was to be compacted by a sheepsfoot roller, the random impervious sections by twin rollers, and the pervious sections by a plain cylindrical roller. A minimum of six passes of the rollers was specified.

*Test results are shown in Exhibit C

Material for rock toes and riprap were to be obtained primarily from structural excavations. The rock was to be dumped in place with the larger rocks at the outer faces and the smaller rocks and spalls adjacent to the embankment. Gravel bedding and select gravel filter material was to be inspected visually.

Stability analyses of the embankment itself yielded a factor of safety near 2.0 for the most severe loading conditions, i.e., sudden drawdown. Also, owing to the presence of the weaker foundation soils, a stability analysis against sliding was conducted, and yield a factor of safety of 1.87. This value is based on a conservative estimate of ϕ = 25 degrees.

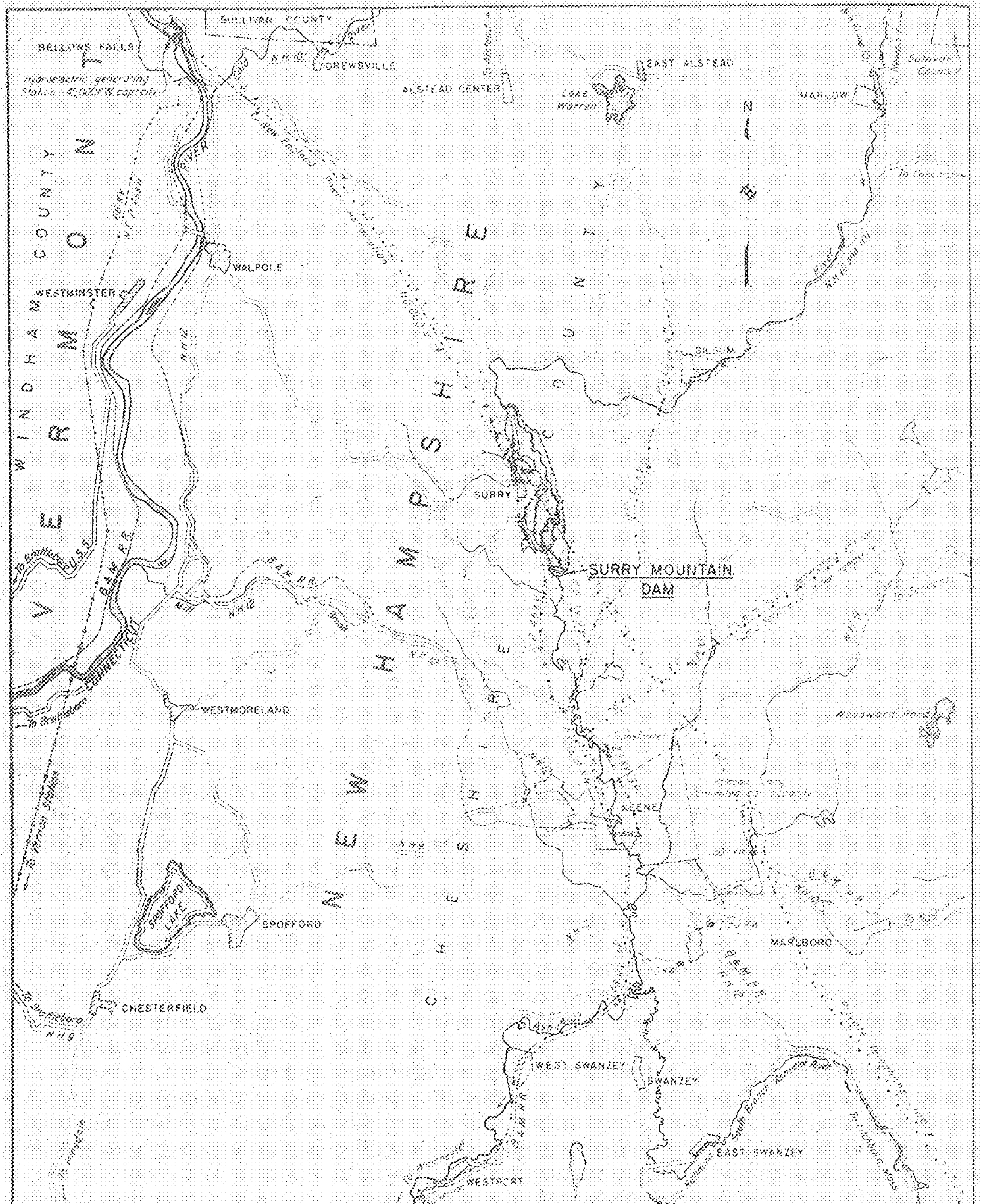
6. CONCLUSIONS AND RECOMMENDATIONS

The potentials for actual liquefaction and cyclic mobility of the embankment soils are expected to be "negligible," as these soils will dilate during shear. Although the embankment soils might exhibit cyclic mobility, the strains are expected to be small because of the high density of these materials and the relatively low (0.13g) peak ground acceleration.

Standard penetration resistances and the results of monotonic and cyclic triaxial tests of foundations sands and silts* indicate the presence of foundation soil zones with "possible" potential for liquefaction and a cyclic mobility potential of "high" at the embankment toes and "possible" at the embankment centerline. It is recommended that additional explorations and laboratory testing be conducted to better establish those soil properties related to liquefaction and cyclic mobility and the spatial distribution of those properties. It is also recommended that a dynamic analysis be performed which properly accounts for seismic loading and material properties.

*Test results are shown in Exhibit C

WAR DEPARTMENT | CORPS OF ENGINEERS, U.S. ARMY



VICINITY MAP

SCALE

2 MILES

LEGEND

- HIGHWAYS
- RAILROADS
- ELECTRIC POWER LINE
- MAXIMUM RESERVOIR FLOW LINE
- ELECTRIC SUBSTATION
- HYDROELECTRIC GENERATING STATION

INDEX TO DRAWINGS

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2	HYDROGRAPHS
3	PLAN OF SUBSURFACE EXPLORATIONS NO 1
4	PLAN OF SUBSURFACE EXPLORATIONS NO 2
5	RECORD OF SUBSURFACE EXPLORATIONS NO 1
6	RECORD OF SUBSURFACE EXPLORATIONS NO 2
7	RECORD OF SUBSURFACE EXPLORATIONS NO 3
8	WORK AREA
9	GENERAL PLAN
10	EMBANKMENT DETAILS NO 1
11	EMBANKMENT DETAILS NO 2
12	SPILLWAY - PLAN AND SECTIONS
13	SPILLWAY - APPROACH AND WEIR
14	RETAINING WALLS
15	OUTLET WORKS
16	ASSEMBLY - GATE SHAFT AND TUNNEL
17	INTAKE STRUCTURE - DETAILS NO 1
18	INTAKE STRUCTURE - DETAILS NO 2
19	OUTLET PORTAL
20	TUNNEL TRANSITION
21	GATE SHAFT AND BASEMENT
22	GATE SHAFT DETAILS
23	BASEMENT DETAILS
24	INTAKE STRUCTURE - STEEL REINFORCEMENT NO 1
25	INTAKE STRUCTURE - STEEL REINFORCEMENT NO 2
26	OUTLET PORTAL AND SPILLWAY STRUCTURES - STEEL REINFORCEMENT
27	TUNNEL TRANSITION - STEEL REINFORCEMENT NO 1
28	TUNNEL TRANSITION - STEEL REINFORCEMENT NO 2
29	GATE SHAFT AND BASEMENT - STEEL REINFORCEMENT NO. 1
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31	BASEMENT - STEEL REINFORCEMENT NO 1
32	BASEMENT - STEEL REINFORCEMENT NO 2
33	STEEL REINFORCEMENT - BAR SCHEDULE NO 1
34	STEEL REINFORCEMENT - BAR SCHEDULE NO. 2
35	STEEL REINFORCEMENT - BAR SCHEDULE NO 3
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37	GATES AND ACCESSORIES - GENERAL ARRANGEMENT NO 1
38	GATES AND ACCESORIES - GENERAL ARRANGEMENT NO 2
39	GATES AND ACCESSORIES - CONDUIT LINING NO 1
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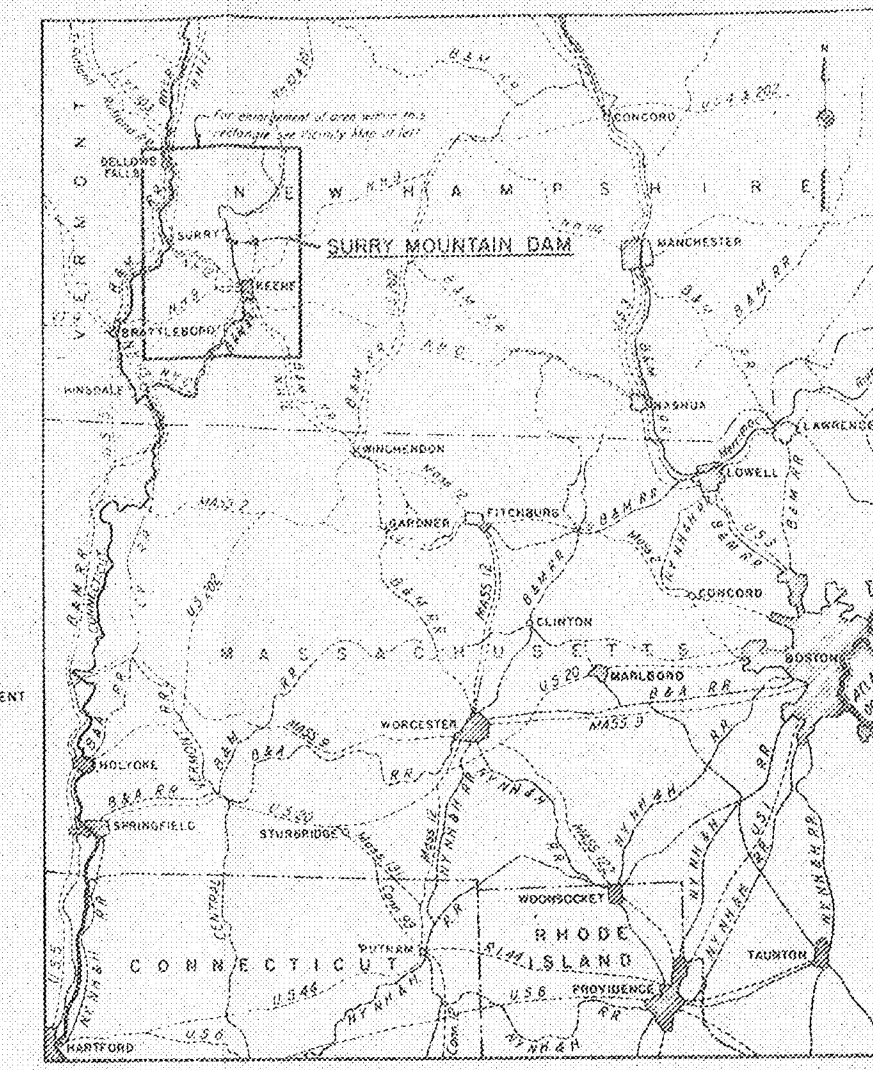
48	OPERATING HOUSE - STRUCTURAL STEEL
49	ELECTRIC LIGHT AND POWER SYSTEM
50	GAGES - TILE AND STAFF
51	ACCESS ROAD
52	ACCESS BRIDGE - SUBSTRUCTURE
53	ACCESS BRIDGE - SUPERSTRUCTURE NO 1
54	ACCESS BRIDGE - SUPERSTRUCTURE NO 2
△ 55	FIELD OFFICE
△ 55	ACCESS ROAD - WING WALL AND TOE WALL
△ 56	INCOMING POWER SERVICE CABLE
△ 57	ACCESS ROAD - EAST ABUTMENT



LOCATION MAP

SCALE

10 0 10 20 MILES

LEGEND

- HIGHWAYS
- RAILROADS

KEY	DATE	REVISION	REV. BY	CK. BY	AP. BY
△	1/2/20	Sheet No. 57 added to set	[illegible]	[illegible]	[illegible]
△	9/14/41	Sheet No. 56 added to set	[illegible]	[illegible]	[illegible]
△	5/9/41	Sheet No. 55 added to set	[illegible]	[illegible]	[illegible]
△	8/29/39	Field Office drawing omitted	[illegible]	[illegible]	[illegible]

CONNECTICUT RIVER FLOOD CONTROL

SURRY MOUNTAIN DAM

PROJECT LOCATION AND INDEX

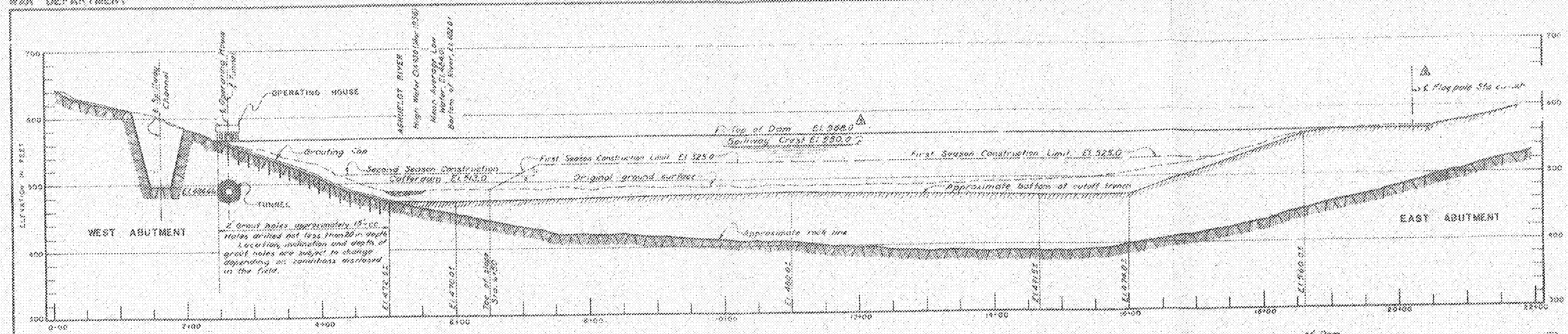
ASHUELOT RIVER, NEW HAMPSHIRE

IN 57 SHEETS | SCALE AS SHOWN | SHEET NO. 1

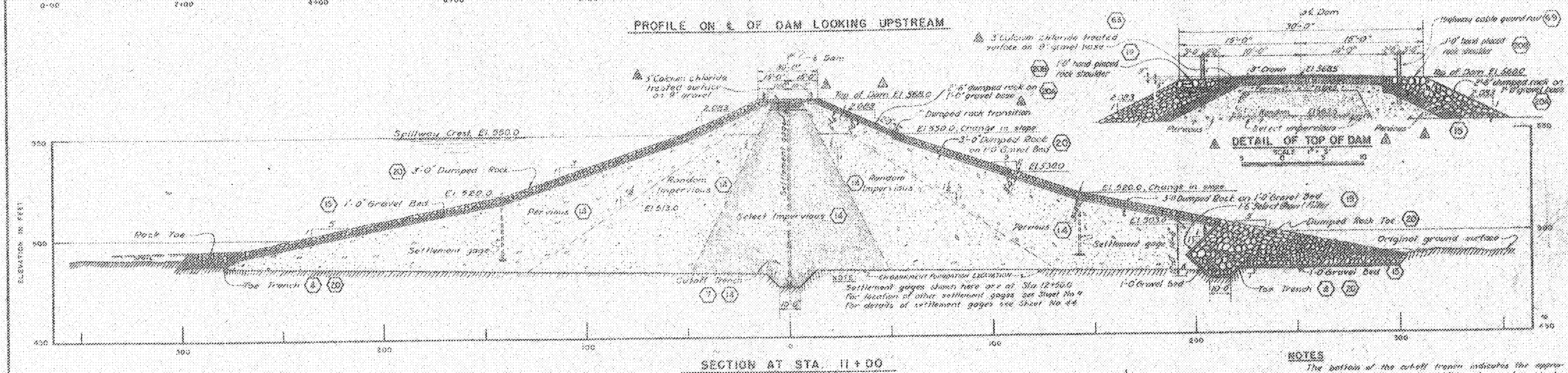
U. S. ENGINEER OFFICE, PROVIDENCE, R. I. DEC. 1938

SUBMITTED: T. D. Burns, PRINCIPAL ENGINEER, CHIEF, ENGINEERING SECTION | APPROVAL RECOMMENDED: R. C. York, CAPT. CORPS OF ENGINEERS, CHIEF, FLOOD CONTROL DIVISION | APPROVED: [illegible], COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

DESIGNED: [illegible] | DRAWN BY W. L. H. TRACED BY W. L. H. ICW. CHECKED BY [illegible] | FILE NO. CT-1-1-27

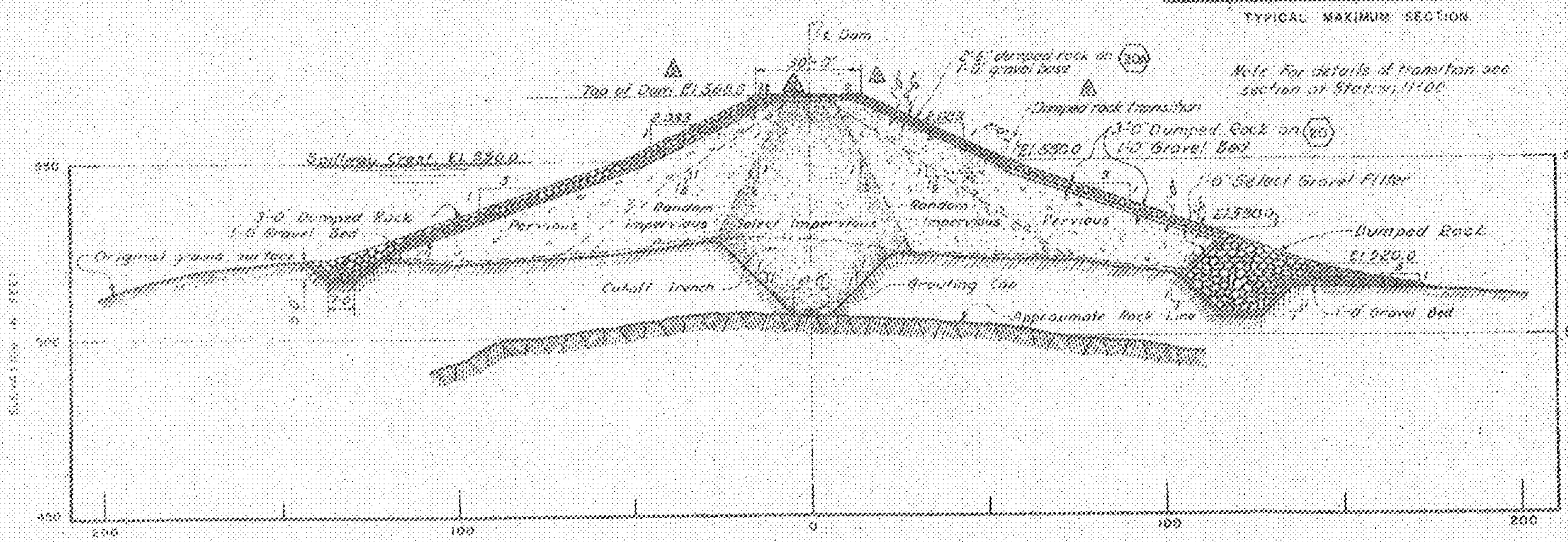


PROFILE ON ℄ OF DAM LOOKING UPSTREAM

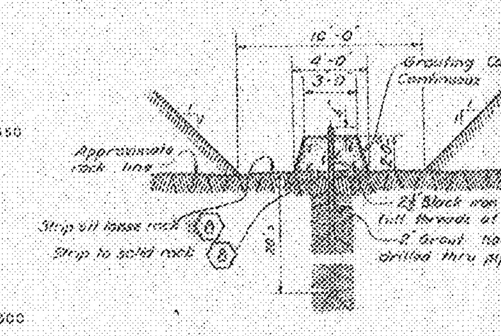


SECTION AT STA. 11+00

TYPICAL MAXIMUM SECTION



SECTION AT STA. 4+00



GROUTING CAP DETAILS

NOTES

The bottom of the cutoff trench indicates the approximate limit of excavation and backfill, and may be raised or lowered depending on actual conditions as disclosed in the field.

Elevations refer to Mean Sea Level Datum.

Sections are taken at right angles to Stations on the centerline of the dam.

See Sheet No. 11 for additional details of embankment.

Figures in hexagons indicate bid item numbers under which payment [illegible] made.

The indicated line of demarcation between the select impervious and random impervious fill, in the embankment, will be variable; the line between the random impervious and pervious fill shall be adhered to.

Both upstream and downstream slopes change from 1 on 2.083 to 1 on 3 at Elev. 550.0 and also from 1 on 3 to 1 on 5 at Elev. 520.0.

Key	Date	Revision indicated by △			
	Sept. 1948	Final field corrections			
△	[illegible]	Road surface revised	[illegible]	[illegible]	[illegible]
		Hand placed shoulder added			
△	[illegible]	Road surface changed	[illegible]	[illegible]	[illegible]
△	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
△	[illegible]	℄ Dam changed. Section added	[illegible]	[illegible]	[illegible]

CONNECTICUT RIVER FLOOD CONTROL

SURRY MOUNTAIN DAM

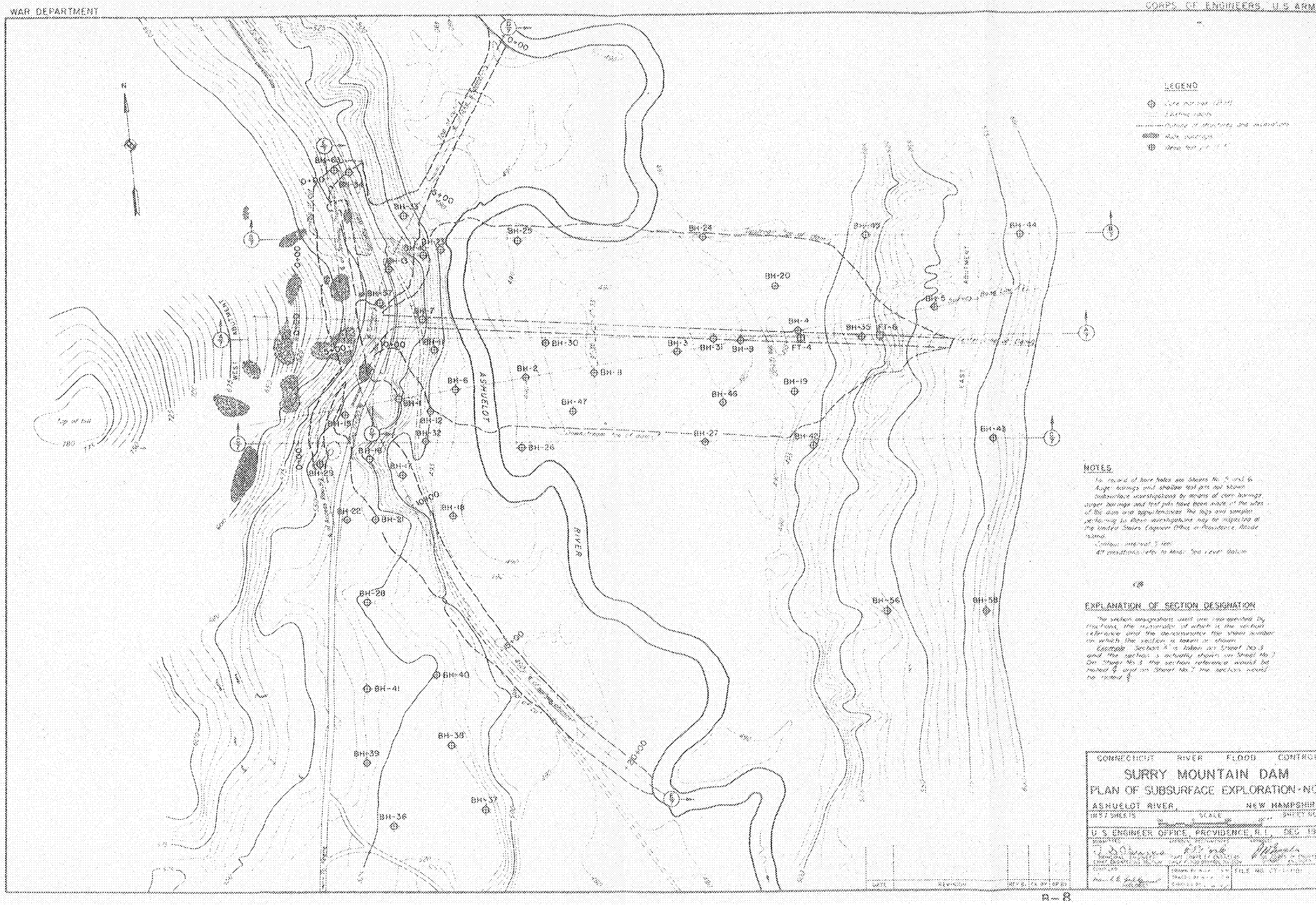
EMBANKMENT DETAILS NO. 1

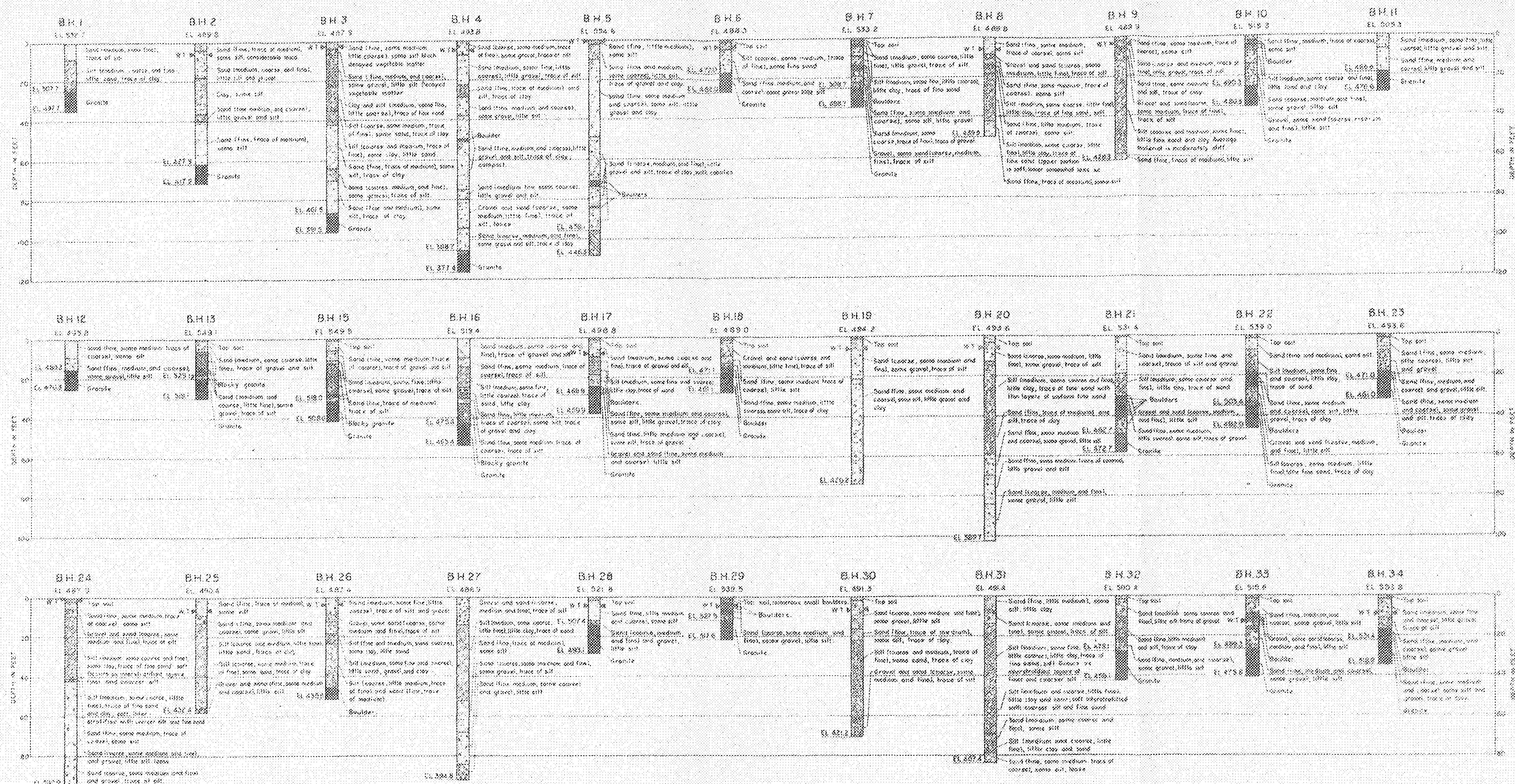
ASHUELOT RIVER, NEW HAMPSHIRE

IN 57 SHEETS — SCALE AS SHOWN — SHEET NO. 10

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., DEC. 1938

FILE NO. CT-1-1231


WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
LEGEND
NOTES
EXPLANATION OF SECTION DESIGNATION
ASHUELOT
RIVER
ABUTMENT
BH-63
BH-33
BH-25
BH-24
BH-49
BH-44
BH-13
BH-20
BH-37
BH-7
BH-5
BH-4
BH-35
FT-6
BH-30
BH-3
BH-31
BH-9
FT-4
BH-2
BH-8
BH-6
BH-19
BH-46
BH-47
BH-12
BH-32
BH-27
BH-42
BH-43
BH-26
BH-29
BH-18
BH-22
BH-21
BH-28
BH-56
BH-58
BH-40
BH-41
BH-38
BH-39
BH-37
BH-36
0+00
5+00
10+00
CONNECTICUT RIVER FLOOD CONTROL
SURRY MOUNTAIN DAM
PLAN OF SUBSURFACE EXPLORATION - NO. 1
ASHUELOT RIVER, NEW HAMPSHIRE
SCALE
U.S. ENGINEER OFFICE, PROVIDENCE, R.I. DEC. 1938



LEGEND

Sand — Silt or Rock flour
Clay — Granite
Gravel — Blocky or Fractured Granite

KEY TO DESCRIPTIONS

MODIFYING TERM	VALUE
And	40% to 50%
Some	20% to 40%
Little	5% to 20%
Trace	5% or less

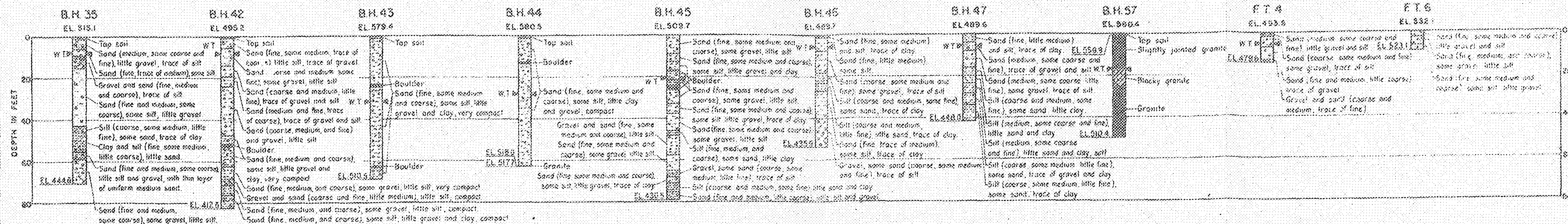
Water table indicated thus W.T.

NOTES

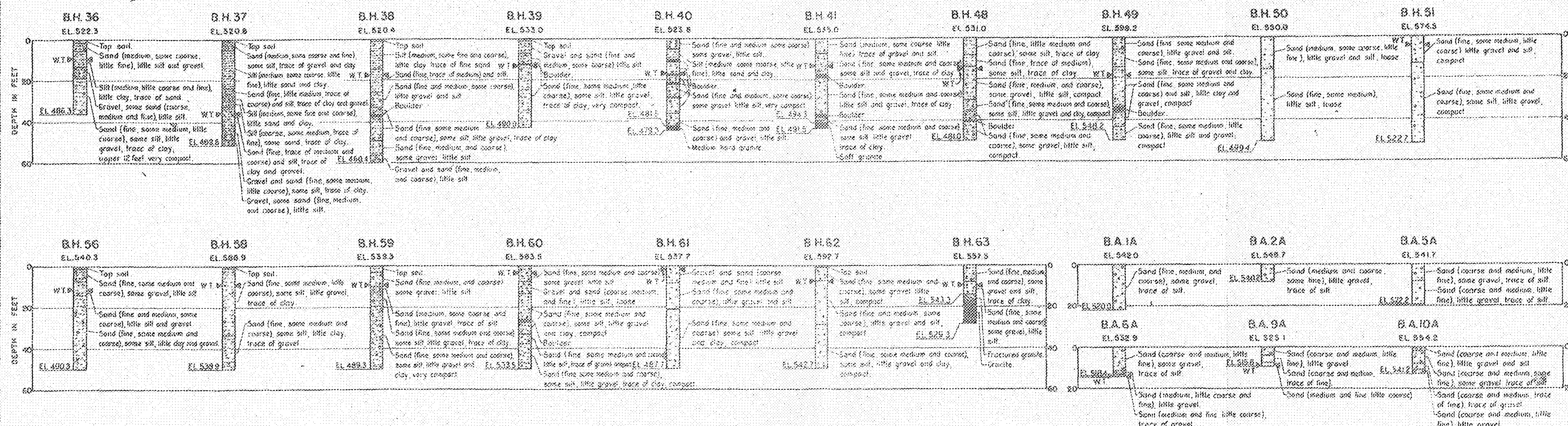
Rock formation is a medium and coarse grained foliated granite.
Water table elevations indicated on this sheet although derived from the best available data, should be regarded as approximate only and subject to considerable variation.
Elevations refer to Mean Sea Level Datum.
For location of bore holes see Sheet No. 3.
B.H. = Core Boring

CONNECTICUT RIVER FLOOD CONTROL
SURRY MOUNTAIN DAM
RECORD OF SUBSURFACE EXPLORATION - NO. 1
ASHUELOT RIVER — NEW HAMPSHIRE
IN 57 SHEETS — SCALE AS SHOWN — SHEET NO. 9
U.S. ENGINEER OFFICE, PROVIDENCE, R.I. — DEC 1938
FILE NO. CT-1-1-83

DATE — REVISION



FOUNDATION EXPLORATIONS



BORROW AREA EXPLORATIONS

LEGEND

- Sand
- Silt or Rock flour
- Clay
- Granite
- Gravel
- Blocky or Fractured Granite

KEY TO DESCRIPTIONS

MODIFYING TERM	VALUE
And	40% to 50%
Some	20% to 40%
Little	5% to 20%
Trace	5% or less

Water table indicated thus W.T.

NOTES

For general notes applying to this sheet, see Sheet No. 5.
For location of Foundation Explorations, see Sheet No. 3.
For location of Borrow Area Explorations, see Sheet No. 4.
B.H. - Core Boring
B.A. - Auger Boring
F.T. - Test Pit

DATE	REVISION	REV BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

SURRY MOUNTAIN DAM

RECORD OF SUBSURFACE EXPLORATION - NO. 2

ASHUELOT RIVER, NEW HAMPSHIRE

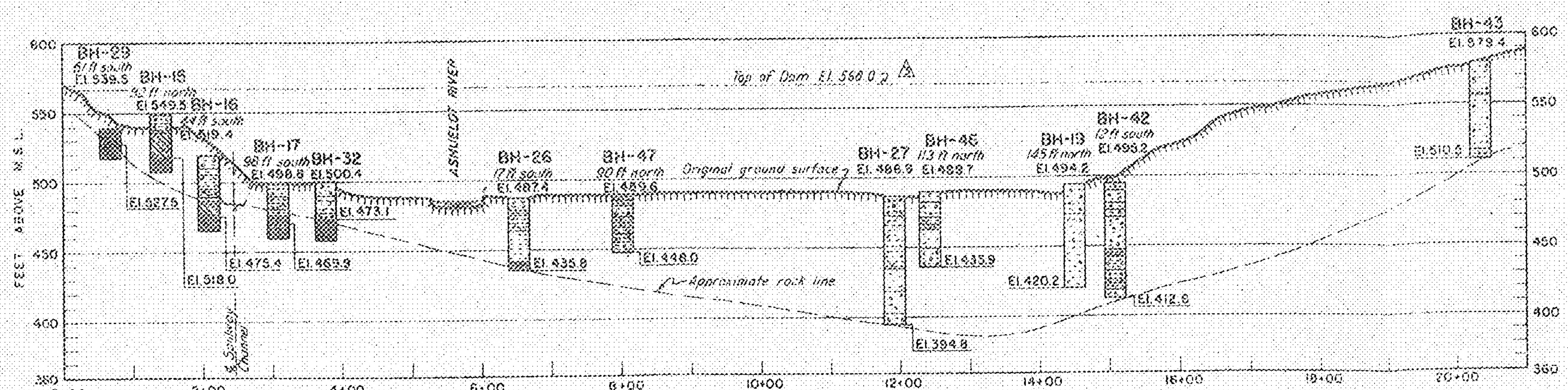
IN 57 SHEETS — SCALE AS SHOWN — SHEET NO. 6

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., DEC. 1938

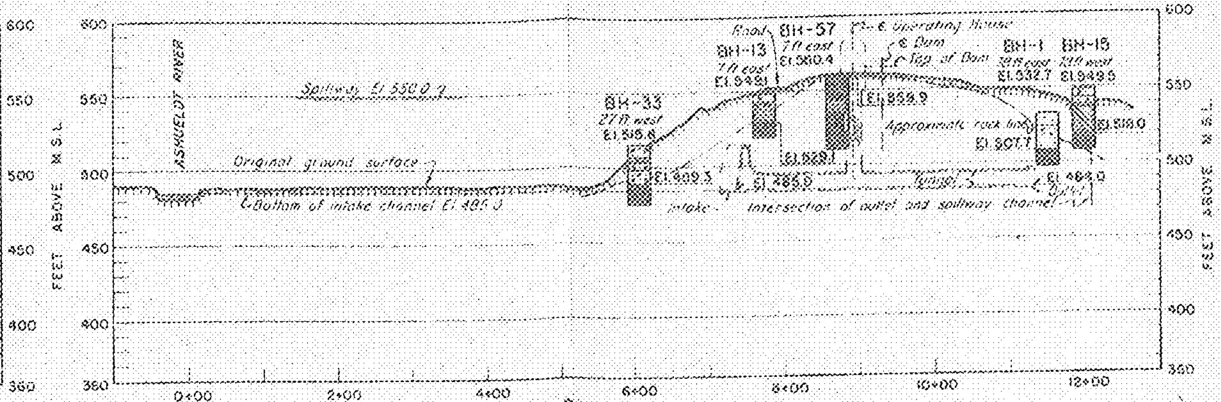
SUBMITTED: PRINCIPAL ENGINEER, CHIEF, ENGINEERING SECTION
APPROVAL RECOMMENDED: CAPT. CORPS OF ENGINEERS, CHIEF, FLOOD CONTROL DIVISION
APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

COMPILED: GEOLOGIST
DRAWN BY W.L.H.
TRACED BY W.L.H.
CHECKED BY

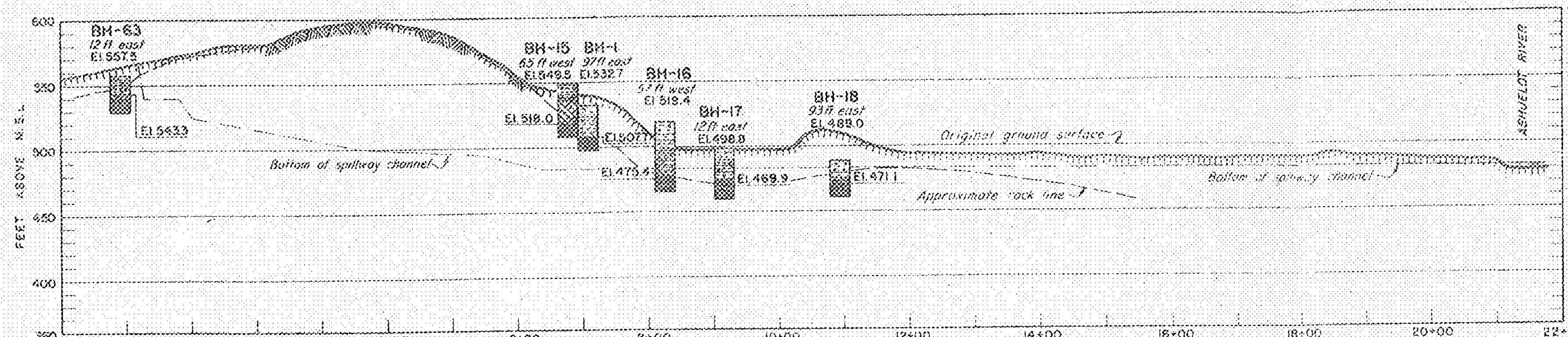
FILE NO. CT-1-1184



SECTION ON LINE $\frac{C}{3}$ - LOOKING UPSTREAM



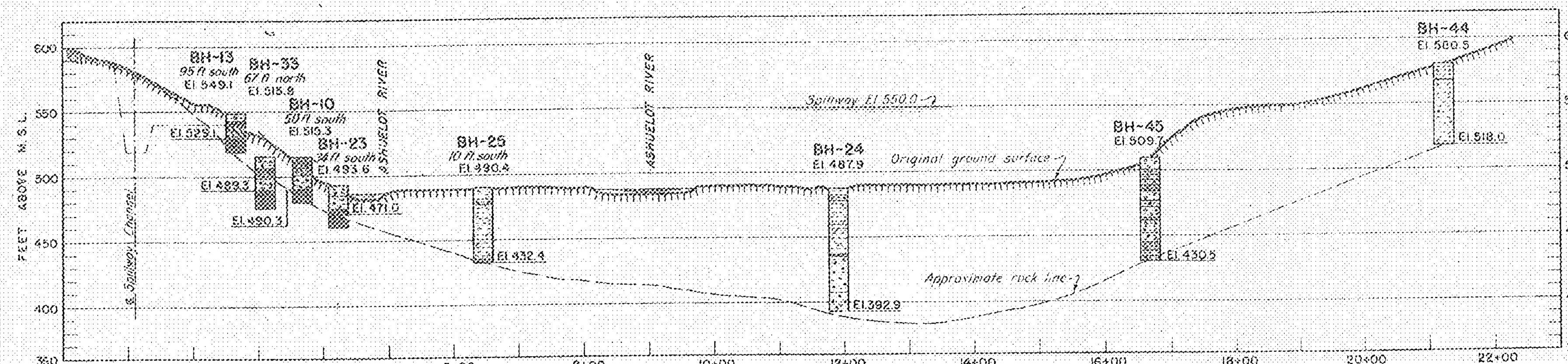
PROFILE ON LINE $\frac{D}{3}$ - CENTER LINE OF TUNNEL



PROFILE ON LINE $\frac{S}{3}$ CENTER LINE OF SPILLWAY CHANNEL LOOKING TOWARD DAM

LEGEND

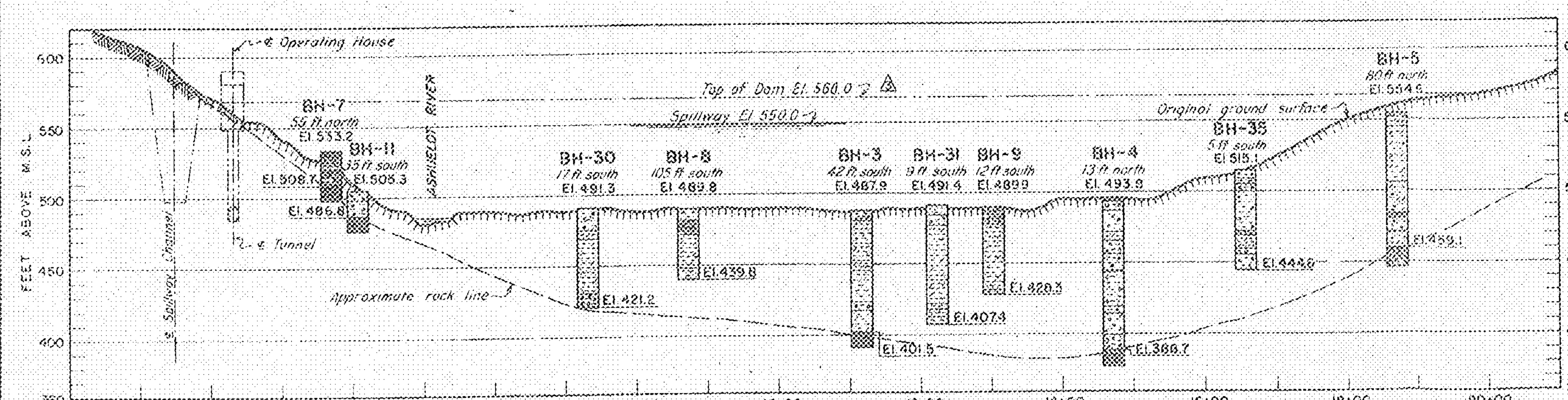
- Sand
- Silt or Rock flour
- Clay
- Granite
- Gravel
- Blocky or Fractured Granite



SECTION ON LINE $\frac{B}{3}$ - LOOKING UPSTREAM

NOTES

For location of profiles and sections see Sheet No 3
For location of bore holes see Sheet No. 3
For logs of bore holes see Sheets No. 5 and 6
Elevation refer to Mean Sea Level Datum
Center lines of graphic logs coincide with those of bore holes



SECTION ON LINE $\frac{A}{3}$ - LOOKING UPSTREAM

KEY	DATE	REVISION	REV BY	CK BY	AP BY
△	1-4-41	Elevations changed. Indicated by △1			

CONNECTICUT RIVER FLOOD CONTROL

SURRY MOUNTAIN DAM

RECORD OF SUBSURFACE EXPLORATION - NO. 3

ASHUELOT RIVER, NEW HAMPSHIRE

IN 57 SHEETS — SCALE AS SHOWN — SHEET NO. 7

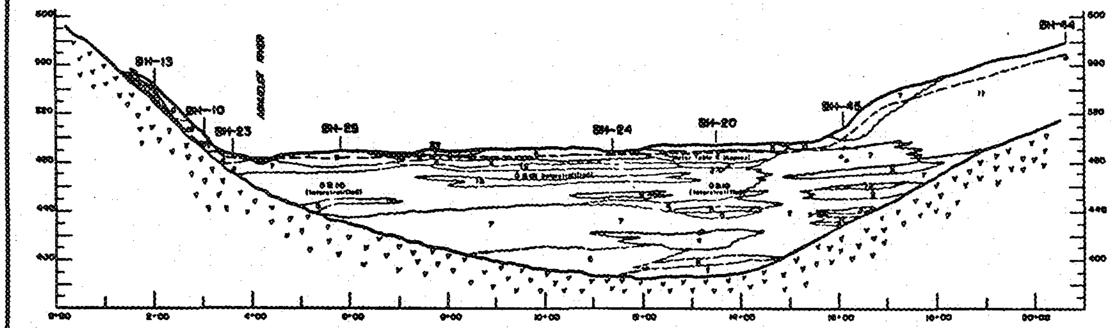
U. S. ENGINEER OFFICE, PROVIDENCE, R.I., DEC. 1938

SUBMITTED: PRINCIPAL ENGINEER, CHIEF ENGINEERING SECTION
APPROVAL RECOMMENDED: CAPT. CORPS OF ENGINEERS, CHIEF, FLOOD CONTROL DIVISION
APPROVED: COL. CORPS OF ENGINEERS, DISTRICT ENGINEER

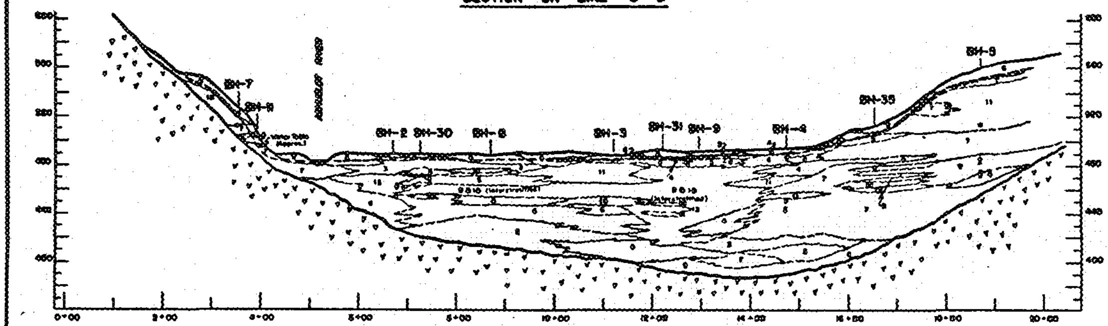
COMPILED: GEOLOGIST
DRAWN BY R.J.K.
TRACED BY
CHECKED BY
FILE NO. CT-1-1165

WAR DEPARTMENT

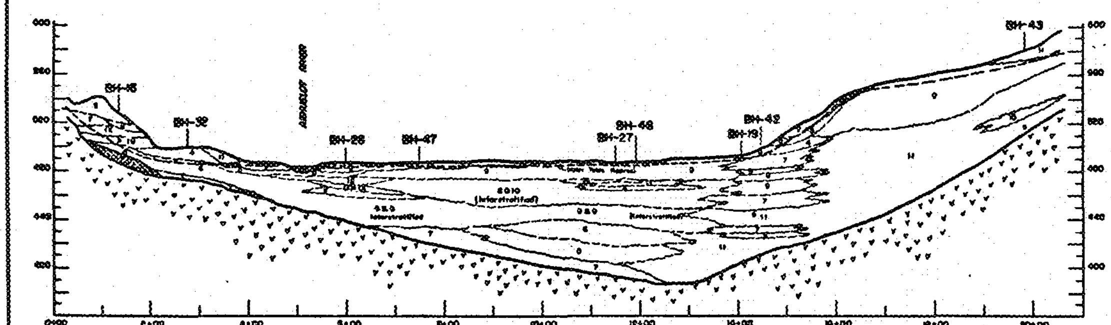
CORPS OF ENGINEERS, U.S. ARMY



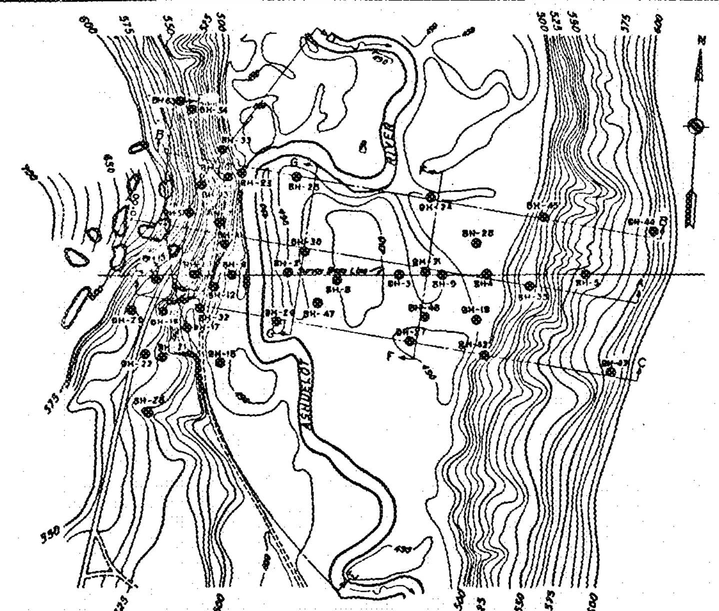
SECTION ON LINE B-B



SECTION ON LINE A-A



SECTION ON LINE C-C



PLAN OF BORE HOLES

SCALE

LEGEND

1	Clean Gravel Contains little coarse to medium Sand.
2	Uniform Coarse to Medium Sand. Contains little gravel and fine sand.
3	Variable - Graded from Gravel to Medium Sand. Contains little fine sand.
4	Uniform Medium to Fine Sand. Contains little coarse sand and coarse silt.
5	Variable - Graded from Gravel to Fine Sand. Contains little coarse silt.
6	Uniform Fine Sand to Coarse Silt. Contains little medium sand and medium silt.
7	Variable - Graded from Gravel to Coarse Silt. Contains little medium silt.
8	Uniform Coarse to Medium Silt. Contains little fine sand and fine silt.
9	Variable - Graded from Gravel to Medium Silt. Contains little fine silt.
10	Uniform Medium to Fine Silt. Contains little coarse silt and coarse clay. Possesses behavior characteristics of silt.
11	Variable - Graded from Gravel or Coarse Sand to Fine Silt. Contains little coarse clay.
12C	Uniform Medium Silt to Coarse Clay. Contains little coarse silt and medium clay. Possesses behavior characteristics of clay.
13	Variable - Graded from Coarse Sand to Clay. Contains little fine clay (colloids). Possesses behavior characteristics of silt.
14	Uniform Fine Silt to Medium Clay. Contains little medium silt and fine clay (colloids). Possesses behavior characteristics of silt.
13C	Variable Clay. Graded from Sand to Fine Clay (colloids). Possesses behavior characteristics of Clay.
14C	Uniform Clay. Contains little silt. Possesses behavior characteristics of clay.

NOTES

Elevations refer to Mean Sea Level Datum.
Numbers in Legend refer to Soil Classes adopted by Providence District.

Blocky condition in granite.

Granite

DATE	REVISION	REV BY	CK BY	AP BY

CONNECTICUT RIVER FLOOD CONTROL

SURRY MOUNTAIN DAM
GEOLOGIC SECTIONS

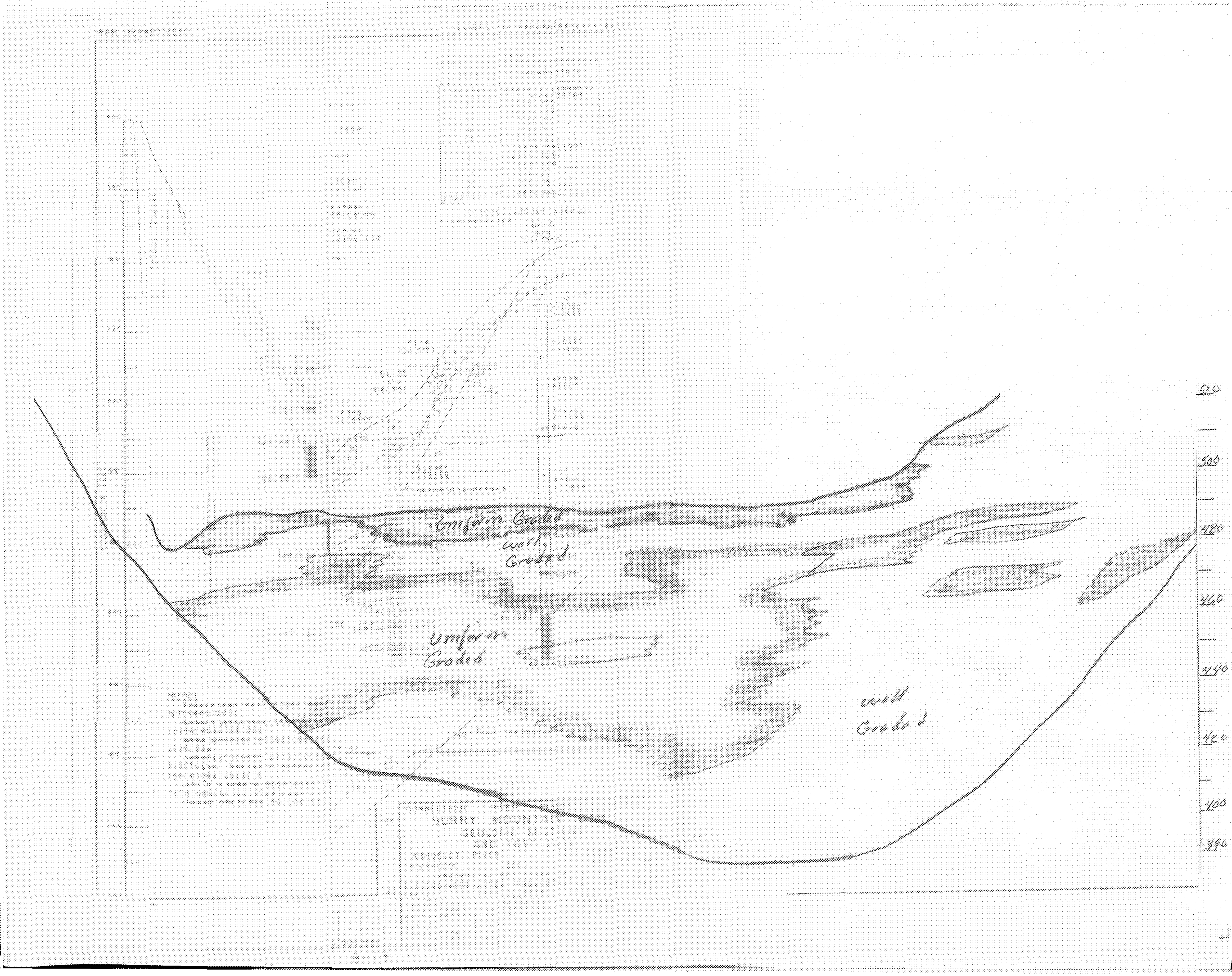
ASHUELOT RIVER — NEW HAMPSHIRE

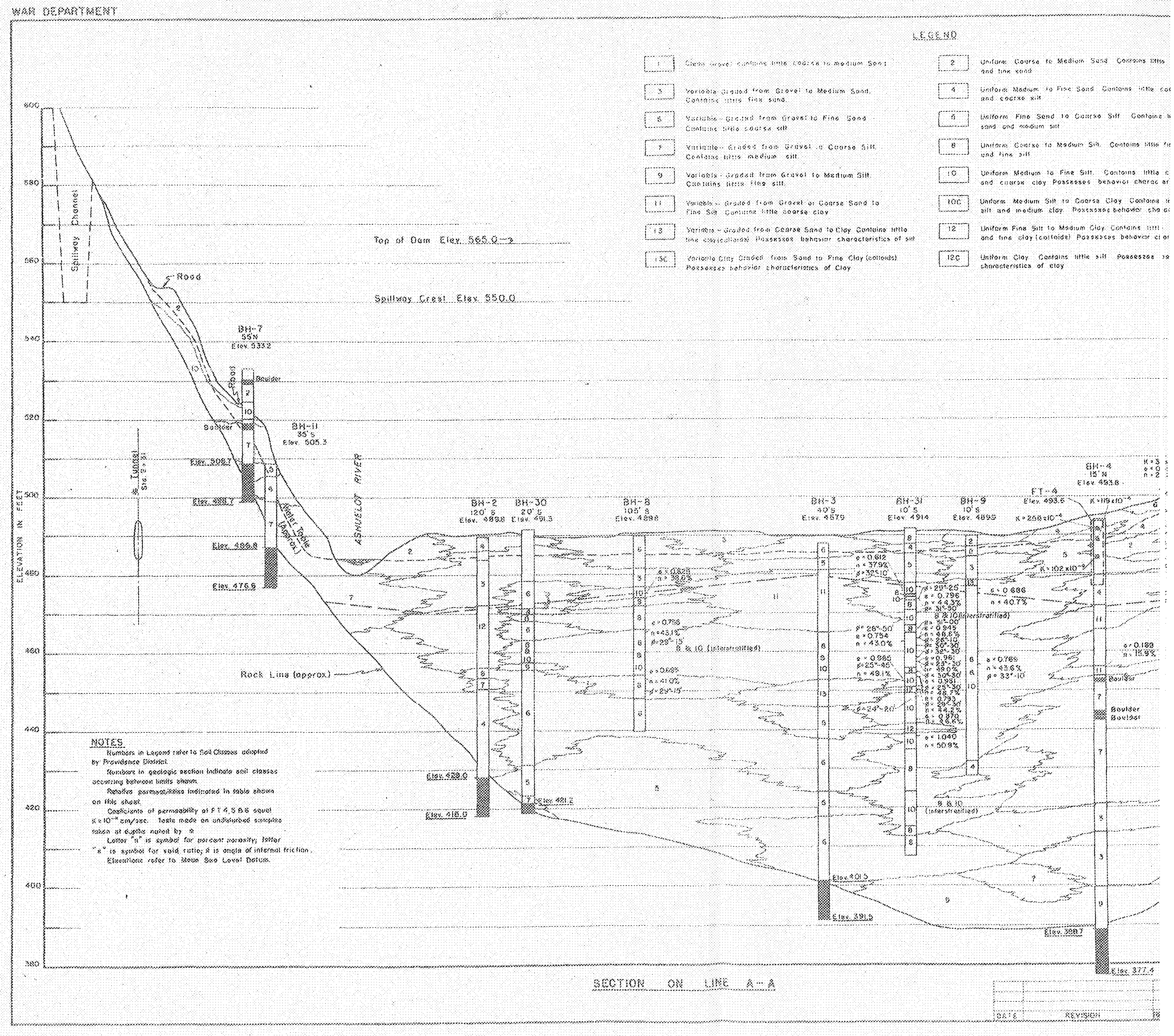
IN 3 SHEETS — SCALES AS SHOWN — SHEET NO. 1

U.S. ENGINEER OFFICE, PROVIDENCE, R.I., MAY 1938

SUBMITTED: T. J. Byrne, PRINCIPAL ENGINEER, CHIEF ENGINEERING SECTION
APPROVAL RECOMMENDED: CAPT. CORPS OF ENGINEERS, CHIEF FLOOD CONTROL DIVISION
APPROVED: LT. COL. CORPS OF ENGINEERS, DISTRICT ENGINEER
COMPILED: GEOLOGIST
DRAWN BY:
TRACED BY:
CHECKED BY:
FILE NO. CT-2-1081

R - 17


WAR DEPARTMENT
CORPS OF ENGINEERS U.S. ARMY
NOTES
Uniform Graded
well Graded
Uniform Graded
well Graded
CONNECTICUT RIVER
SURRY MOUNTAIN
GEOLOGIC SECTIONS
AND TEST DATA
ASHUELOT RIVER
U.S. ENGINEER OFFICE
520
500
480
460
440
420
400
390
B-13


WAR DEPARTMENT
LEGEND
1 Clean Gravel contains little coarse to medium Sand
2 Uniform Course to Medium Sand. Contains little and fine sand
3 Variable Graded from Gravel to Medium Sand. Contains little fine sand.
4 Uniform Medium to Fine Sand. Contains little co and coarse silt
5 Variable - Graded from Gravel to Fine Sand. Contains little coarse silt
6 Uniform Fine Sand to Coarse Silt. Contains li sand and medium silt
7 Variable - Graded from Gravel to Coarse Silt. Contains little medium silt.
8 Uniform Coarse to Medium Silt. Contains little fi and fine silt
9 Variable - Graded from Gravel to Medium Silt. Contains little fine silt.
10 Uniform Medium to Fine Silt. Contains little c and coarse clay Possesses behavior charac er
11 Variable - Graded from Gravel or Coarse Sand to Fine Silt. Contains little coarse clay
10C Uniform Medium Silt to Coarse Clay. Contains li silt and medium clay. Possesses behavior cha c
13 Variable - Graded from Coarse Sand to Clay. Contains little fine clay (colloids) Possesses behavior characteristics of silt
12 Uniform Fine Silt to Medium Clay. Contains littl and fine clay (colloids) Possesses behavior c or
13C Variable Clay Graded from Sand to Fine Clay (colloids) Possesses behavior characteristics of Clay
12C Uniform Clay. Contains little silt. Possesses be characteristics of clay
NOTES
Numbers in Legend refer to Soil Classes adopted by Providence District.
Numbers in geologic section indicate soil classes occurring between limits shown.
Relative permeabilities indicated in table shown on this sheet.
Coefficients of permeability of F.T. 4, 5 & 6 equal K×10⁻⁴ cm/sec. Tests made on undisturbed samples taken at depths noted by ※
Letter "n" is symbol for percent porosity; letter "e" is symbol for void ratio; ø is angle of internal friction.
Elevations refer to Mean Sea Level Datum.
ELEVATION IN FEET
Top of Dam Elev. 565.0
Spillway Crest Elev. 550.0
Spillway Channel
Road
Rock Line (approx.)
Water Table (Approx.)
ASHUELOT RIVER
BH-7 55'N Elev. 533.2
BH-11 35'S Elev. 505.3
BH-2 120'S Elev. 489.8
BH-30 20'S Elev. 491.3
BH-8 105'S Elev. 489.8
BH-3 40'S Elev. 487.9
BH-31 10'S Elev. 491.4
BH-9 10'S Elev. 489.5
FT-4 Elev. 493.5
BH-4 15'N Elev. 493.8
Boulder
SECTION ON LINE A-A
DATE
REVISION

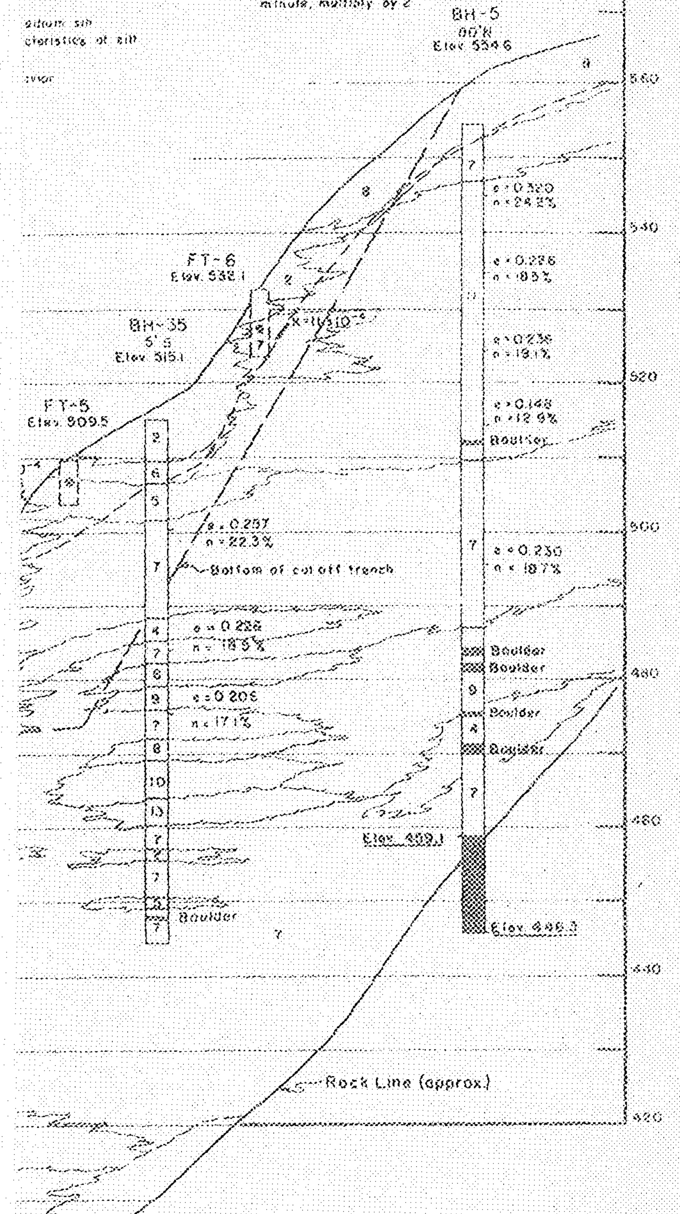
CORPS OF ENGINEERS, U.S. ARMY

TABLE

RELATIVE PERMEABILITIES	
Class of Material	Coefficient of Permeability $k \times 10^{-4}$ cm/sec
2	120 to 400
4	20 to 120
6	5 to 20
8	1 to 5
10	0.1 to 1.0
1	Greater than 1000
3	200 to 1000
5	50 to 200
7	15 to 50
9	3 to 15
11	0.2 to 3.0

NOTE

To convert coefficient to foot per minute, multiply by 2



CONNECTICUT RIVER FLOOD CONTROL
SURRY MOUNTAIN DAM
GEOLOGIC SECTIONS
AND TEST DATA
ASHUELOT RIVER NEW HAMPSHIRE

IN 3 SHEETS SCALE SHEET NO. 3
HORIZONTAL 1 IN. = 50 FT. — VERTICAL 1 IN. = 10 FT.

U.S. ENGINEER OFFICE, PROVIDENCE, R.I. MAY, 1938

SUBMITTED
PRINCIPAL ENGINEER
CHIEF ENGINEERING SECTION

APPROVAL RECOMMENDED
CAPT. CORPS OF ENGINEERS
CHIEF FLOOD CONTROL DIVISION

APPROVED
CORPS OF ENGINEERS
DISTRICT ENGINEER

COMPILED
GEOLOGIST

DRAWN BY
TRACED BY
CHECKED BY

FILE NO.

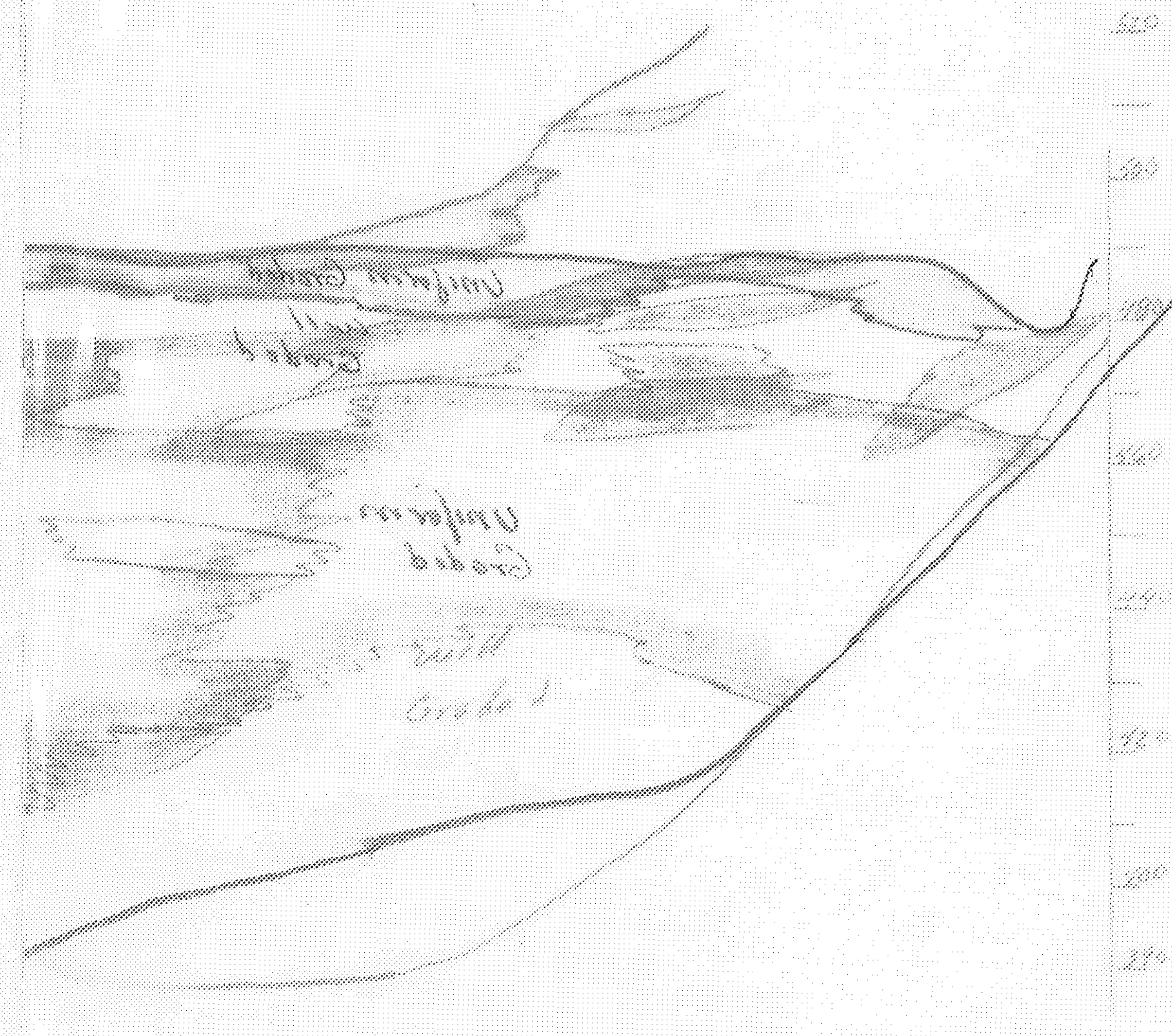
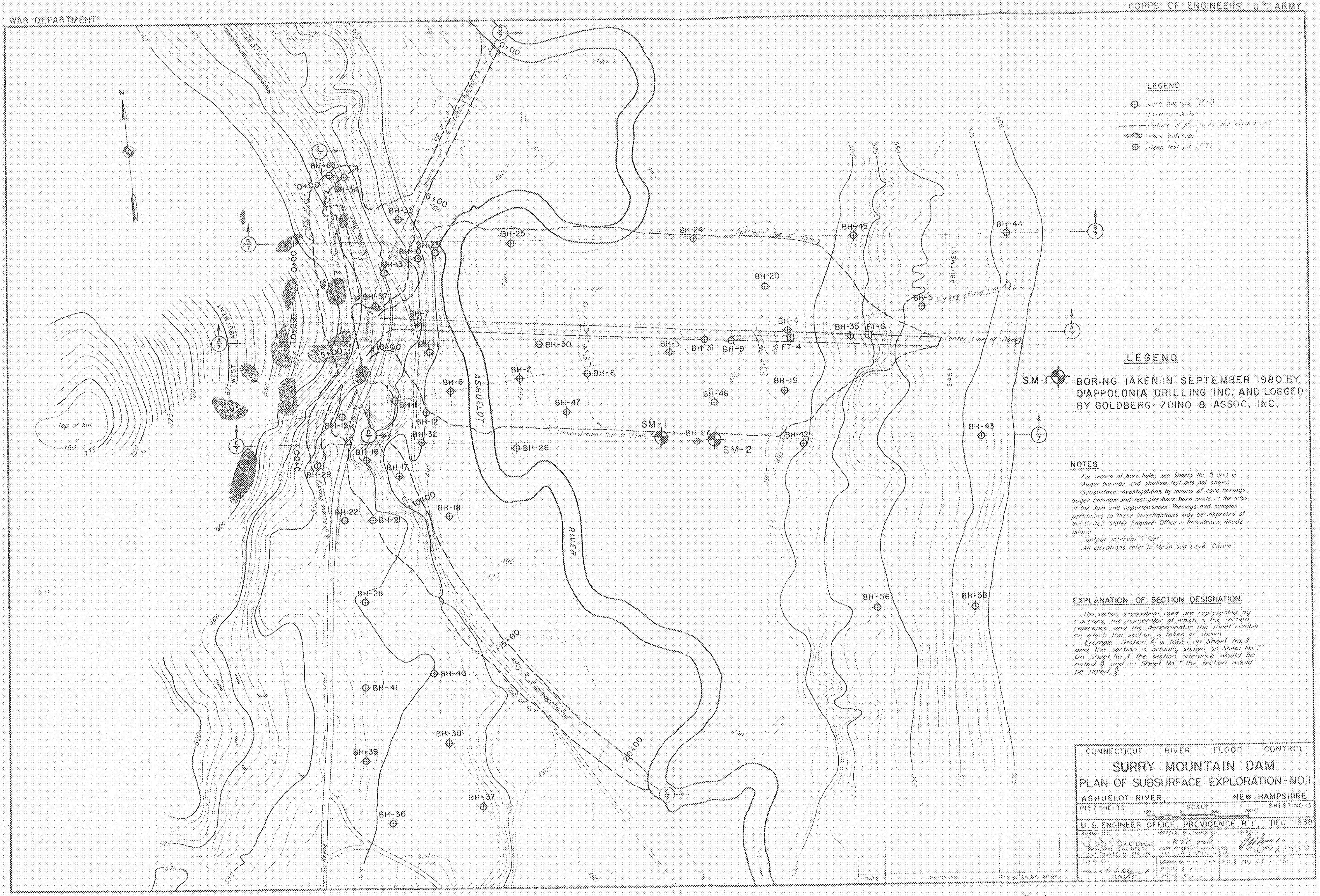


EXHIBIT C

BORING LOGS AND LABORATORY TEST RESULTS*

*The material in this Exhibit was obtained from report entitled "Liquefaction and Cyclic Mobility Potential, Corps of Engineers Completed New England Dams, Phase II - Investigation, February 1981.


WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
LEGEND
SM-1
BORING TAKEN IN SEPTEMBER 1980 BY D'APPOLONIA DRILLING INC. AND LOGGED BY GOLDBERG-ZOINO & ASSOC. INC.
NOTES
Subsurface investigations by means of core borings, auger borings and test pits have been made at the sites of the dam and appurtenances.
Contour interval 5 feet.
EXPLANATION OF SECTION DESIGNATION
ASHUELOT
RIVER
Center Line of Dam
Downstream Toe of Dam
WEST ABUTMENT
EAST ABUTMENT
Top of hill
SM-1
SM-2
FT-4
FT-6
BH-2
BH-3
BH-4
BH-5
BH-6
BH-7
BH-8
BH-9
BH-12
BH-13
BH-15
BH-16
BH-17
BH-18
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BH-29
BH-30
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BH-43
BH-44
BH-45
BH-46
BH-47
BH-56
BH-57
BH-58
BH-60
0+00
5+00
10+00
15+00
20+00
CONNECTICUT RIVER FLOOD CONTROL
SURRY MOUNTAIN DAM
PLAN OF SUBSURFACE EXPLORATION - NO. 1
ASHUELOT RIVER, NEW HAMPSHIRE
IN 57 SHEETS
SCALE
SHEET NO. 3
U.S. ENGINEER OFFICE, PROVIDENCE, R.I. DEC. 1938

GOLDBERG, ZOINO, DUNNICLIFF & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	PROJECT SURRY MOUNTAIN DAM KEENE, NEW HAMPSHIRE	REPORT OF BORING NO. SM-1 SHEET 1 OF 3 DATE 10/1/80 FILE G-2729

BORING CO. D'Appolonia Drilling
FOREMAN Steve Brilmyer
G-Z-D ENGINEER T. vonRosenvinge

BORING LOCATION STA. 11+10 (See Plan)
GROUND ELEV. 489.5'± M.S.L.
DATE START 10/1/80 DATE END 10/3/80

CASING
SIZE: 4" O.D.
HAMMER: N/A lb.
FALL: N/A

SAMPLER
TYPE: 2"O.D. Split Spoon
HAMMER 140 lb.
FALL: 30"
OTHER: 3" Shelby Tube W/ Check Valve

GROUNDWATER READINGS

DATE	DEPTH	CASING AT	STABILIZATION TIME
10/1	4.5'	8'	10 minutes

DEPTH	CAS. BL. /FT.	SAMPLE NO.	PEN./REC.	DEPTH	BLOWS/6"	STRATA CHG. and GEN. DESC.	SAMPLE DESCRIPTION	NOTE
		S-1	18"/12"	3.5'-5.0'	1-1-3	FILL 5'	Loose, gray-brown, fine SAND, some (+) slightly Organic Silt, trace of roots (SM) and (OL)	
5								
		S-2	18"/8"	8.5'-10.0'	6-7-5	GRANULAR FILL 12.5'	Medium dense, brown-gray, fine to coarse SAND, some (-) fine to coarse Gravel, trace (+) Silt, slight organic odor (SW-SM)	
10								
		S-3	18"/10"	13.5'-15.0'	2-1-2	STRATIFIED SILT AND CLAY	Loose, gray, SILT, little fine Sand (ML)	1
15								
		S-4	18"/18"	18.5'-20.0'	1-1-1		Very soft, gray, interbedded SILT and CLAY, trace fine Sand, low to medium plasticity (CL)	
20								
25								
		T-1	24"/24"	26'-28'	Push	29'	Loose, gray SILT and fine SAND (ML)	
30						STRATIFIED FINE SAND AND SILT		
		S-5	18"/10"	30'-31.5'	2-2-2		Loose, gray, fine SAND, trace Silt (SP-SM)	

REMARKS: 1. Hollow stem augering to 15' replaced with 4" O.D. casing to 15' depth, and used bentonite drilling mud beyond this depth.

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. 2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

GOLDBERG, ZOINO, DUNNICLIFF & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

SURRY MOUNTAIN DAM
KEENE, NEW HAMPSHIRE

SHEET 2 OF 3
DATE 10/1/80 FILE G-2729

DEPTH	CAS. BL. /FT.	SAMPLE				STRTA CHNG and GEN DESC.	SAMPLE DESCRIPTION	NOTE
		NO.	PEN./REC.	DEPTH	BLOWS/6"			
35								
		T-2	24"/20"	35'-37'	Push		Gray fine SAND, some Silt (SM)	
40								
		S-6	18"/10"	40'-41.5'	4-5-4	STRATIFIED FINE SAND AND SILT	Loose, gray, fine SAND, little (-) Silt (SP-SM)	
		T-3	24"/24"	43'-45'	Push			
45								
		S-7	18"/12"	48'-49.5'	3-3-3		Loose, gray, SILT and fine SAND, (ML)	
50								
		T-4	24"/22"	53'-55'	Push			
55								
		S-8	18"/8"	58'-59.5'	4-2-6		Loose, gray, fine SAND, little (+) medium Sand layers (1" thick), little Silt, trace Clay layers (1/8" thick) (SP-SM)	
60						61.5'		
		S-9	18"/12"	63'-64.5'	10-9-8	STRATIFIED SAND	Medium dense, gray, fine to coarse SAND, trace (+) Silt (SW-SM)	
65								
		S-10	18"/12"	68'-69.5'	5-7-8		Medium dense, gray, fine SAND, trace Silt, trace (-) fine Gravel (SP-SM)	
70								

REMARKS:

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

DEPTH	CAS. BL. /FT.	SAMPLE NO.	PEN./REC.	DEPTH	BLOWS/6"	STRTA CHNG and GEN. DESC.	SAMPLE DESCRIPTION	NOTE
		S-11	18"/11"	73'-74.5'	15-9-11	Stratified SAND	Medium dense, gray, interbedded layers of: fine to coarse SAND, some Gravel (SW); and fine SAND, little Silt (SP-SM)	
75								
		S-12	18"/14"	78.5'-80'	19-70-19		Very dense, gray, interbedded layers of: fine SAND, little Silt (SP-SM); medium to fine SAND, trace Gravel, trace Silt (SP)	
80								
		S-13	12.5"/10"	82'-83'	29-80-100(1/2")	fractured rock & gravel	Very dense, gray, fine to coarse (+) GRAVEL, some (-) coarse to fine Sand, trace (+) Silt (GP-GM)	2
85							Bottom of hole @83'.	

REMARKS:

2. Refusal - 100 blows with 140 lb. hammer, 1/2 inch penetration.

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

GOLDBERG, ZOINO, DUNNICLIFF & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	PROJECT: SURRY MOUNTAIN DAM, KEENE, NEW HAMPSHIRE	REPORT OF BORING NO. SM-2; SHEET 1 OF 3; DATE 10/3/80; FILE G-2729

BORING CO. D'Appolonia Drilling
FOREMAN Steve Brilmyer
G-Z-D ENGINEER T. vonRosenvinge
BORING LOCATION STA. 12+73 (See Plan)
GROUND ELEV. 490.5'± M.S.L.
DATE START 10/3/80 DATE END 10/4/80

CASING	SAMPLER
SIZE: 6" Hollow Stem Auger	TYPE: 2"O.D. Split Spoon
HAMMER: N/A lb.	HAMMER 140 lb.
FALL: N/A	FALL: 30"
	OTHER:

GROUNDWATER READINGS

DATE	DEPTH	CASING AT	STABILIZATION TIME
10/4	6.5'	15'	14 hours

DEPTH	CAS. BL. /FT.	SAMPLE NO.	PEN./REC.	DEPTH	BLOWS/6"	STRATA CHG. and GEN. DESC.	SAMPLE DESCRIPTION	NOTES
		S-1	18"/18"	3.5'-5.0'	2-1-1	GRANULAR FILL	Loose, brown, fine SAND, little (+) Silt, trace fine roots (moist fill) (SP-SM)	
5								
		S-2	18"/6"	8.5'-10.0'	5-4-2		Loose, gray, fine to coarse SAND, some Gravel, trace Silt, trace organic odor (SW)	
10								
15		S-3	18"/14"	15'-16.5'	8-8-8		Medium dense, brown, fine (+) to coarse, micaceous SAND, some fine Gravel, trace (+) Silt (SW-SM)	
						19'		
20		S-4	18"/18"	21'-22.5'	2-2-3	STRATIFIED SAND, SILT, CLAY	Soft gray SILT, trace thin (1/8" - 1/4" thick) Clay layers, trace fine Sand (ML)	
25		S-5	18"/12"	24.5'-26'	2-2-2		Soft, gray, SILT (ML) and Silty Clay (CL) (clay in layers ranging from 1/4" to 3" thick), trace fine Sand, slight plasticity	
30		S-6	18"/12"	29'-30.5'	6-4-4		Loose, gray, SILT, little fine Sand layers, trace Stiff Clay layers (1/4" thick) (ML)	

REMARKS:

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. 2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF THE GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

DEPTH	CAS. BL. /FT.	SAMPLE NO.	PEN./REC.	DEPTH	BLOWS/6"	STRTA CHNG and GEN DESC.	SAMPLE DESCRIPTION	NOTE
35		S-7	18"/16"	34.5'-36'	2-3-5	STRATIFIED SILT AND CLAY	Loose, gray, clayey SILT, slight plasticity (ML)	1
40		S-8	18"/11"	39.5'-41'	6-4-4		Loose, gray, SILT, some fine Sand trace Stiff Clay (a 1" thick layer) (ML)	
45		S-9	18"/16"	44.5'-46'	3-3-6	46'	Loose, gray SILT, little fine Sand, trace Clay layers (1/8" to 1/2" thick) (ML)	
50		S-10	18"/10"	49.5'-51'	6-6-7	FINE SAND	Medium dense, gray, fine SAND, little (-) Silt (SP-SM)	
55		S-11	18"/14"	54.5'-56'	6-7-9		Medium dense, gray, fine SAND, trace Silt (SP-SM)	
60		S-12	18"/11"	59.5'-61'	4-6-7	62'	Medium dense, gray, fine SAND, little medium Sand layers (1" thick) trace fine Gravel (SP)	
								1
65		S-13	18"/6"	64'-65.5'	12-8-9	STRATIFIED SAND	Medium dense, gray, fine to coarse SAND, some fine Gravel, trace fine Sand layer (1/2" thick) trace Silt (SW-SM)	
70		S-14	18"/8"	69'-70.5'	7-6-7		Medium dense, gray, fine to coarse SAND, little fine Gravel, trace Silt (SW-SM)	

REMARKS: 1. Some gravelly resistance encountered with the roller bit at 37' and from 62' to 64'.

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.
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GOLDBERG, ZOINO, DUNNICLIFF & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	SURRY MOUNTAIN DAM KEENE, NEW HAMPSHIRE	SHEET 3 OF 3 DATE 10/4/80 FILE G-2729

DEPTH	CAS. BL. /FT.	SAMPLE NO.	PEN./REC.	DEPTH	BLOWS/6"	STRTA CHNG and GEN. DESC.	SAMPLE DESCRIPTION	NOTE
75		S-15	18"/1"	74'-75.5'	8-12-10	STRATI-FIED SAND	Medium dense, gray, fine to coarse GRAVEL and SAND, trace Silt (Poor Recovery) (GW)	
80		S-16	18"/10"	79'-80.5'	9-7-8		Medium dense, gray, fine (+) to medium SAND, trace (+) Silt, trace coarse Gravel (SP-SM)	
							Bottom of hole @80.5'	
85								

REMARKS:

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL.
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LABORATORY TEST RESULTS

LABORATORY TESTING DATA SUMMARY

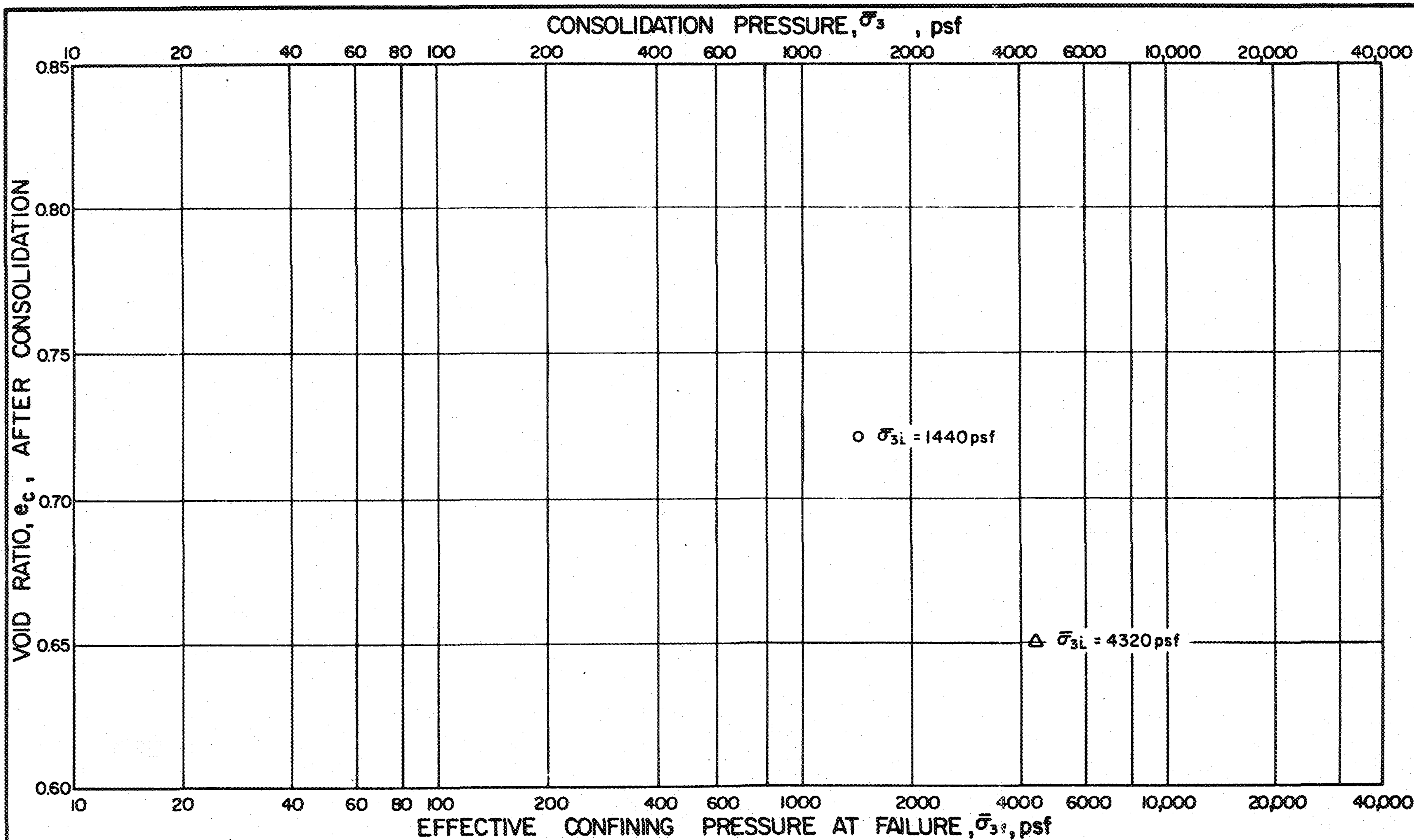
Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS							Perme-ability	STRENGTH TESTS					CONSOL.	Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	G_s	γ_d pcf		Torvane or Type Test	σ_c or $\overline{\sigma}_c$ OR σ psi	Failure Criteria	$\sigma_1 - \sigma_3$ OR τ psi	Strain %	C_c / e_o	
SM1	S3	13.5-15.0	35	35.7														Grey SILT, little fine Sand (ML)
SM1	S4	18.5-20.0	36	34.5	32	22	93	19	2.83									Grey SILT and CLAY of low to medium plasticity, trace fine Sand (CL)
SM1	S5	30.0-31.5	37	23.3														Grey fine SAND, trace Silt (SM - SP)
SM1	S7	48.0-49.5	38	31.6			65.0											Grey SILT and fine SAND (ML)
SM1	S10	68.0-69.5	39	24.6														Grey fine SAND, trace fine Gravel, trace Silt (SM - SP)

LABORATORY TESTING DATA SUMMARY

				IDENTIFICATION TESTS								STRENGTH TESTS					CONSOL.	
Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	G_s	γ_d pcf	Permeability	Torvane or Type Test	σ_c or $\bar{\sigma}_c$ OR σ psi	Failure Criteria	$\sigma_1 - \sigma_3$ OR τ psi	Strain %	C_c / e_o	Laboratory Log and Soil Description
SM1	S12	78.5-80.0	40	18.4														Grey fine to medium SAND, little fine Gravel, trace Silt (SM - SP)
SM2	S4	21.0-22.5	41	37.7														Grey SILT, trace fine Sand (ML)
SM2	S5	24.5-26.0	42	35.3	29	26	97	13										Grey Clayey SILT of slight plasticity. Trace fine Sand (ML)
SM2	S7	34.5-36.0	43	41.2	33	30	100	4										Grey Clayey SILT of slight plasticity (ML)
SM2	S9	44.5 46.0	44	35.4			88											Grey SILT, little fine Sand (ML)

LABORATORY TESTING DATA SUMMARY

Boring No.	Sample No.	Depth ft.	Laboratory or Test No.	IDENTIFICATION TESTS							Permeability	STRENGTH TESTS					CONSOL.	Laboratory Log and Soil Description
				Water Content %	LL %	PL %	Sieve -200 %	Hyd -2μ %	G_s	γ_d pcf		Torvane or Type Test	σ_c or $\bar{\sigma}_c$ OR σ psi	Failure Criteria	$\sigma_1 - \sigma_3$ OR τ psi	Strain %	C_c / e_o	
SM2	S11	54.5-56.0	45	26.8														Grey brown fine SAND, trace Silt (SM - SP)
SM2	S13	64.0-65.5	46	14.1														Grey fine to coarse SAND, some fine Gravel, trace Silt (SM - SW)
SM2	S14	69.0-70.5	47	14.9														Grey fine to coarse SAND, little fine Gravel, trace Silt (SM - SW)

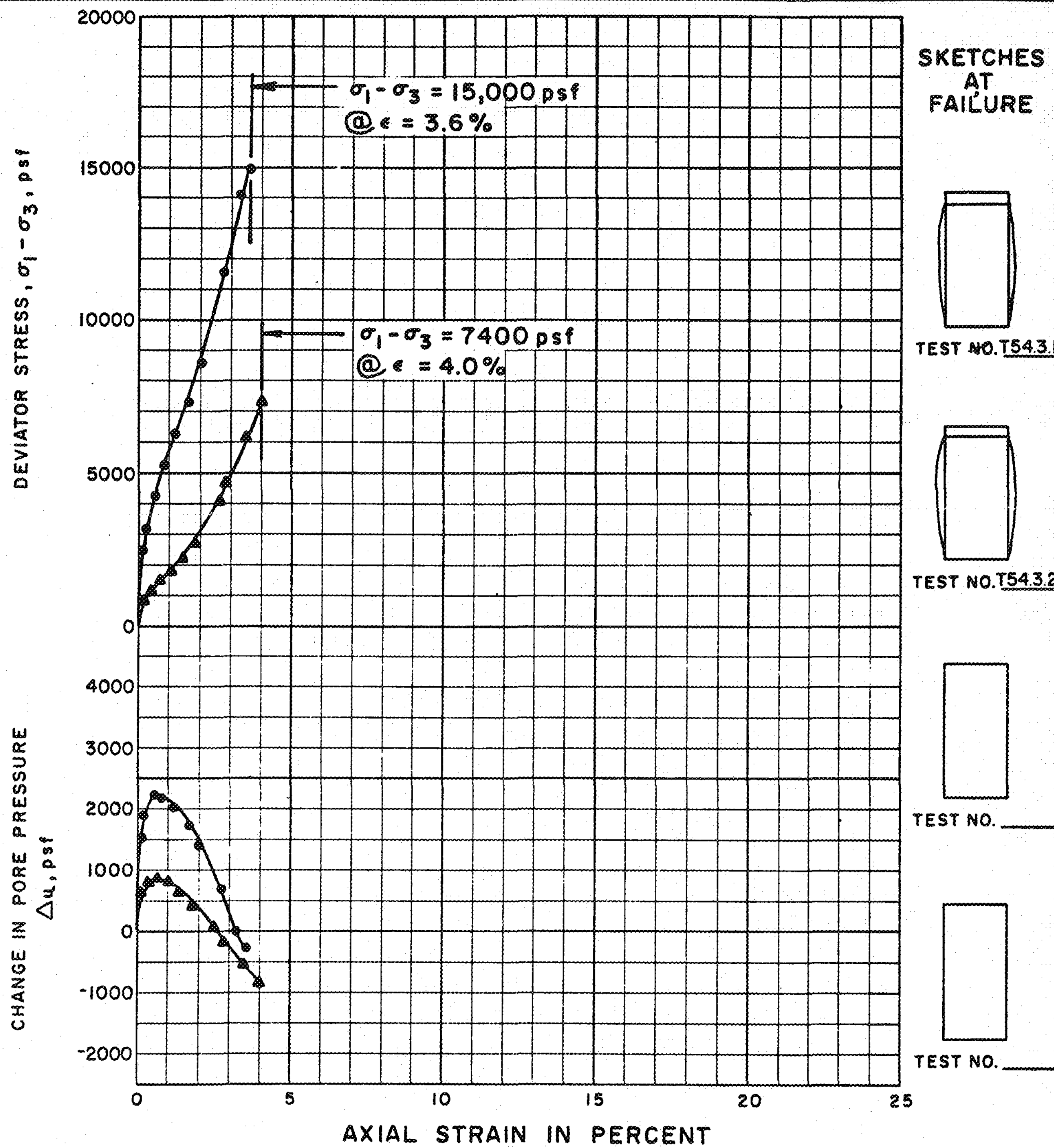


TEST No.	SYM.	BORING No.	DEPTH (ft)	INITIAL CONDITIONS			CONDITIONS BEFORE LOADING					FINAL CONDITIONS	
				ω_N (%)	γ_d pcf	HT. / DIA.	$\bar{\sigma}_1 = \bar{\sigma}_3$ psf	u_b psf	ϵ_v (%)	β (%)	e_c	ω_N (%)	γ_d pcf
T54.3.1	△	SM1	35.9-36.4	22.2	101.7	6.000 / 2.870	4320	10080	1.07	95	0.65	23.4	102.8
T54.3.2	○	SM1	35.4-35.9	24.5	97.8	6.000 / 2.850	1440	7200	0.73	97	0.72	24.5	98.5

SURRY MOUNTAIN DAM
SURRY, N.H.

SUMMARY PLOT
MONOTONIC TRIAXIAL TESTS

DATE DEC. 1980



TEST NO./SYMBOL	INITIAL CONDITIONS			CONDITIONS BEFORE SHEAR				FINAL CONDITIONS		RATE OF STRAIN, PERCENT PER MINUTE
	INITIAL WATER CONTENT, %	INITIAL DRY UNIT WEIGHT, pcf	SAMPLE HEIGHT & DIAMETER, in.	INITIAL STRESSES $\bar{\sigma}_1 = \bar{\sigma}_3$, psf	FINAL BACK PRESSURE, psf	VOLUMETRIC STRAIN, %	PORE PRESSURE RESPONSE, %	FINAL WATER CONTENT, %	FINAL DRY UNIT WEIGHT, pcf	
T54.3.1 ●	22.2	101.7	6.000 2.870	4320	10080	1.07	95	23.4	102.8	-
T54.3.2 ▲	24.5	97.8	6.000 2.850	1440	7200	0.73	97	24.5	98.5	-

SOIL DESCRIPTION: GREY FINE SAND, SOME SILT (SM)

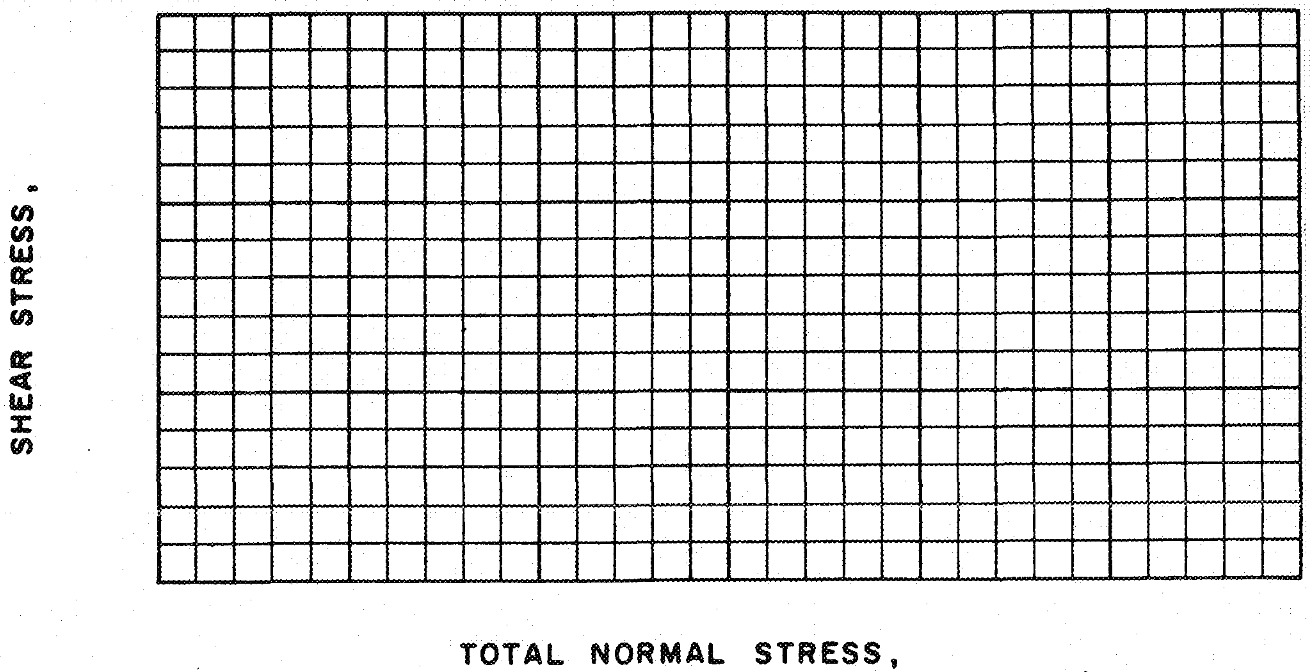
LIQUID LIMIT ____ % PLASTIC LIMIT ____ % SPECIFIC GRAVITY ____

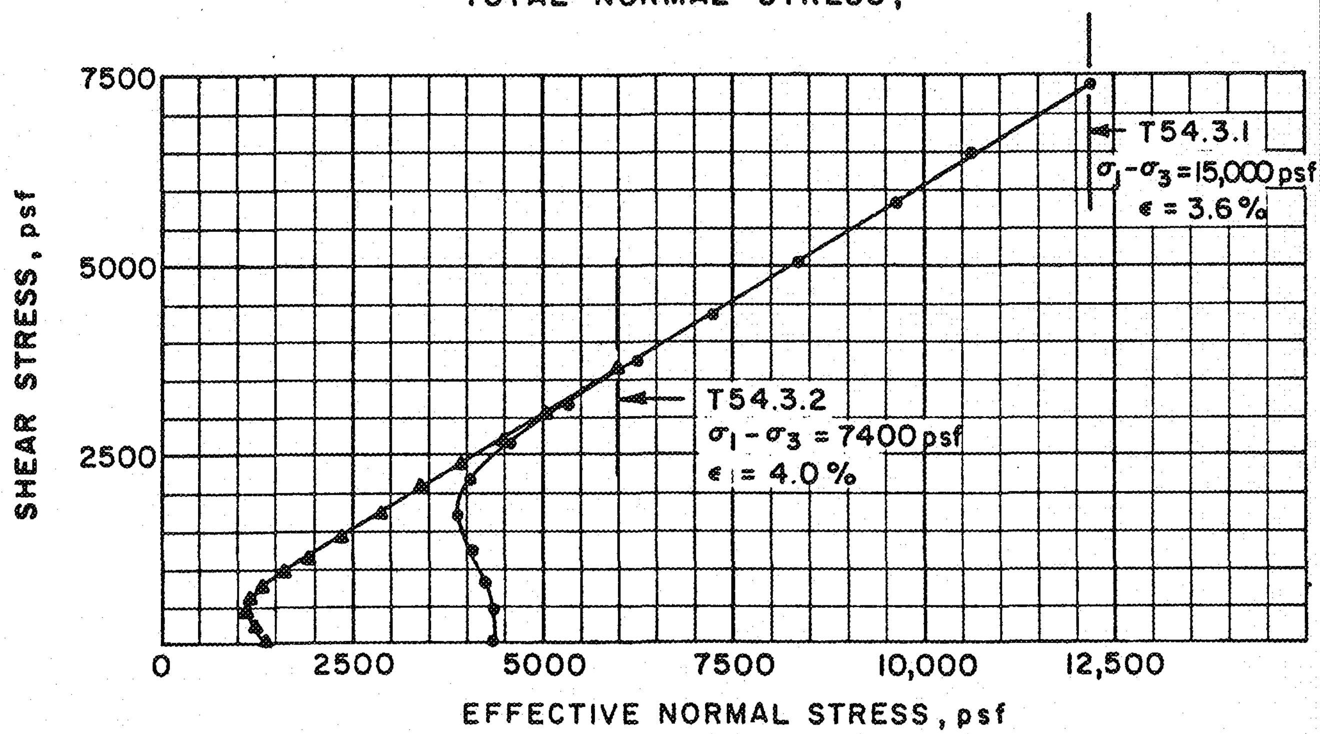
SURRY MOUNTAIN DAM
SURRY, N.H.

TRIAXIAL COMPRESSION TESTS (MONOTONIC)

BORING NO. SM1
SAMPLE T2
DEPTH 35.4'-36.4'

TEST SERIES NO. 54
DATE DEC. 1980





SOIL DESCRIPTION: GREY FINE SAND, SOME SILT

LIQUID LIMIT ____ % PLASTIC LIMIT ____ % SPECIFIC GRAVITY ____

FAILURE CRITERIA ____

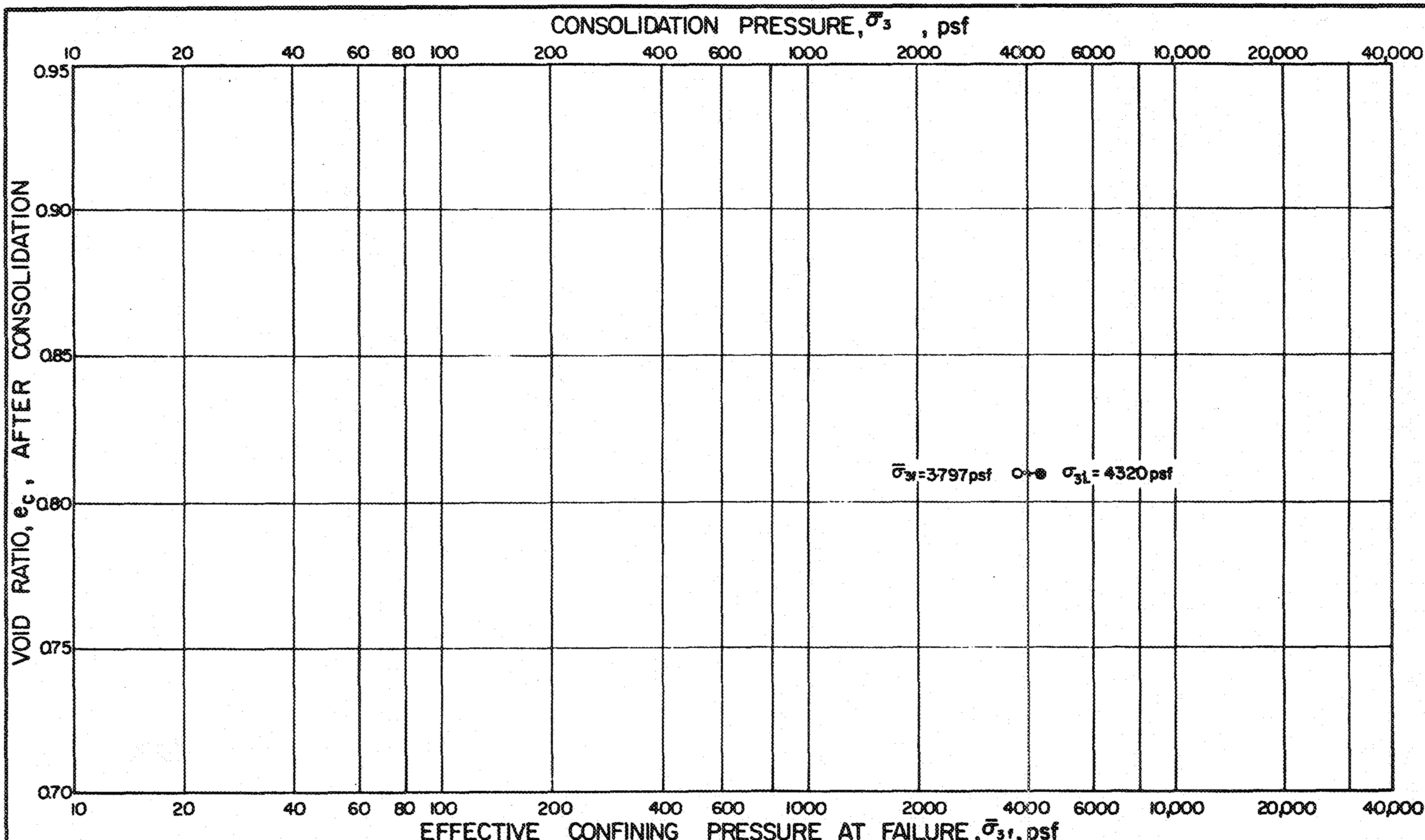
REMARKS ____

SURRY MOUNTAIN DAM
SURRY, N.H.

MOHR STRENGTH ENVELOPE

TRIAXIAL COMPRESSION TESTS (MONOTONIC)

BORING NO. SM1 TEST SERIES NO. 54
SAMPLE T2
DEPTH 35.4'-36.4' DATE DEC. 1980

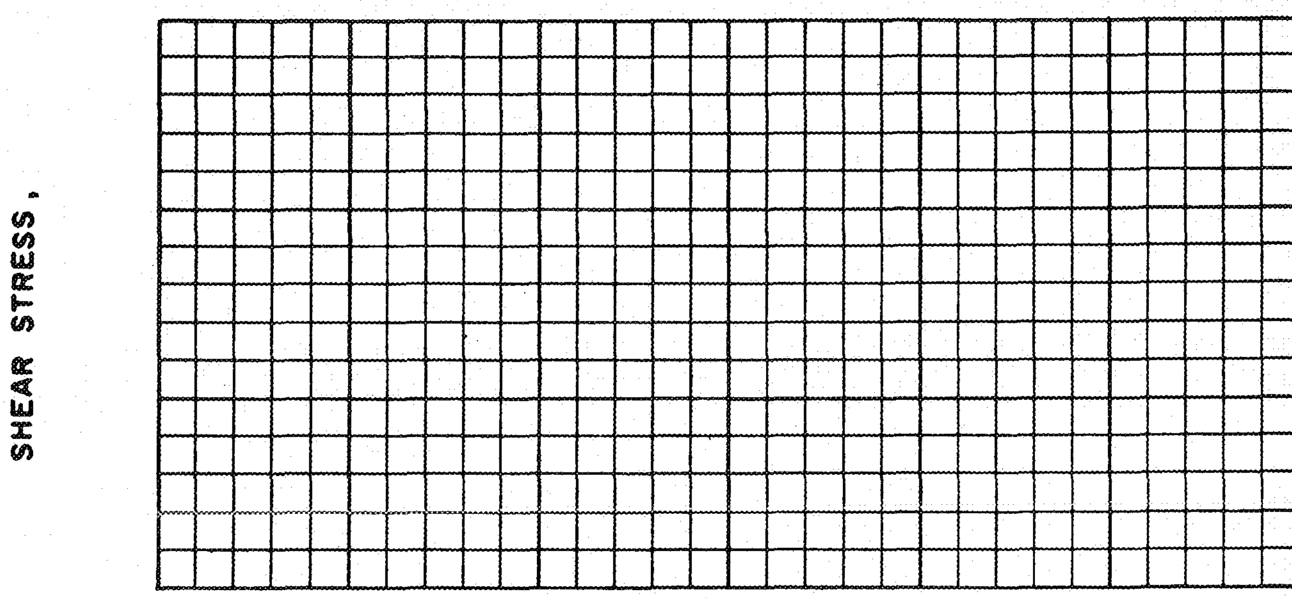


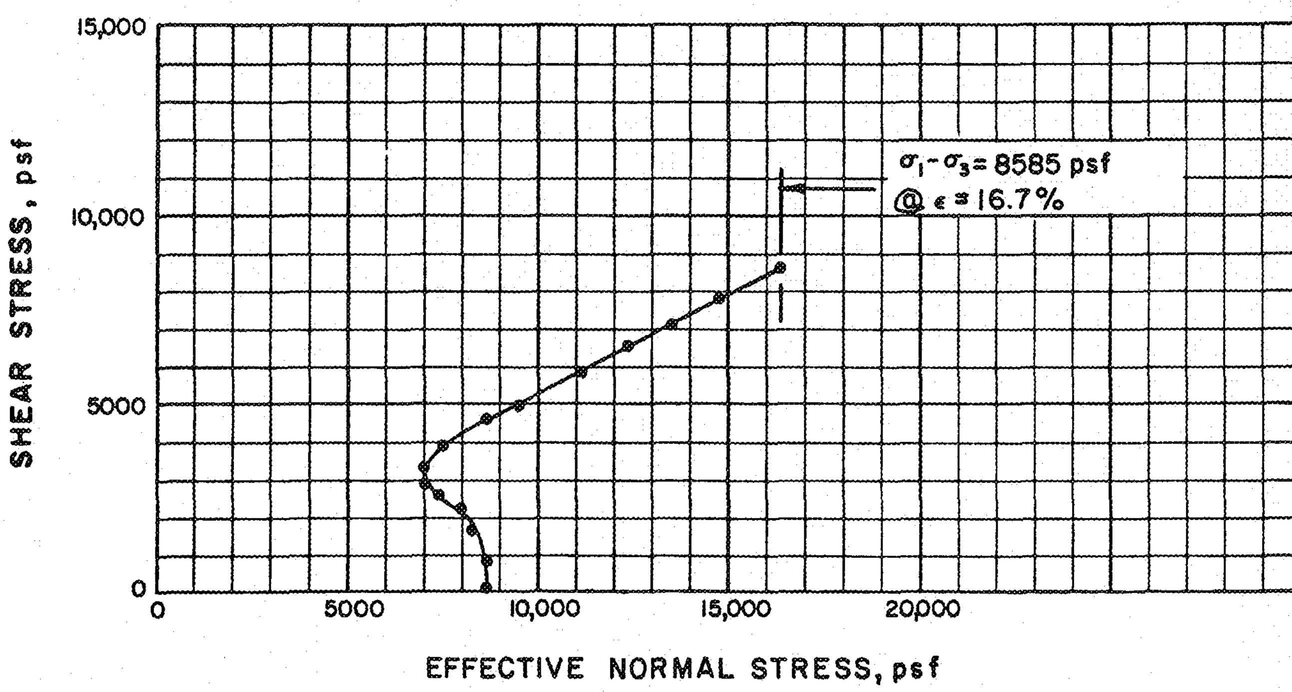
TEST No.	SYM.	BORING No.	DEPTH (ft.)	INITIAL CONDITIONS			CONDITIONS BEFORE LOADING					FINAL CONDITIONS	
				ω_N (%)	γ_d pcf	HT. / DIA.	$\bar{\sigma}_1 = \bar{\sigma}_3$ psf	u_b psf	ϵ_v (%)	β (%)	e_c	ω_N (%)	γ_d pcf
T53.3.2	⊗	SMI	26.3-27.7	6.7	92.2	6.000 / 2.900	4320	8640	1.51	95	0.81	31.5	93.6

SURRY MOUNTAIN DAM
SURRY, N.H.

SUMMARY PLOT
MONOTONIC TRIAXIAL TESTS

DATE DEC. 1980





SOIL DESCRIPTION: GREY SILT AND FINE SAND (ML)

LIQUID LIMIT ____ % PLASTIC LIMIT ____ % SPECIFIC GRAVITY ____

FAILURE CRITERIA ____

REMARKS ____

SURRY MOUNTAIN DAM

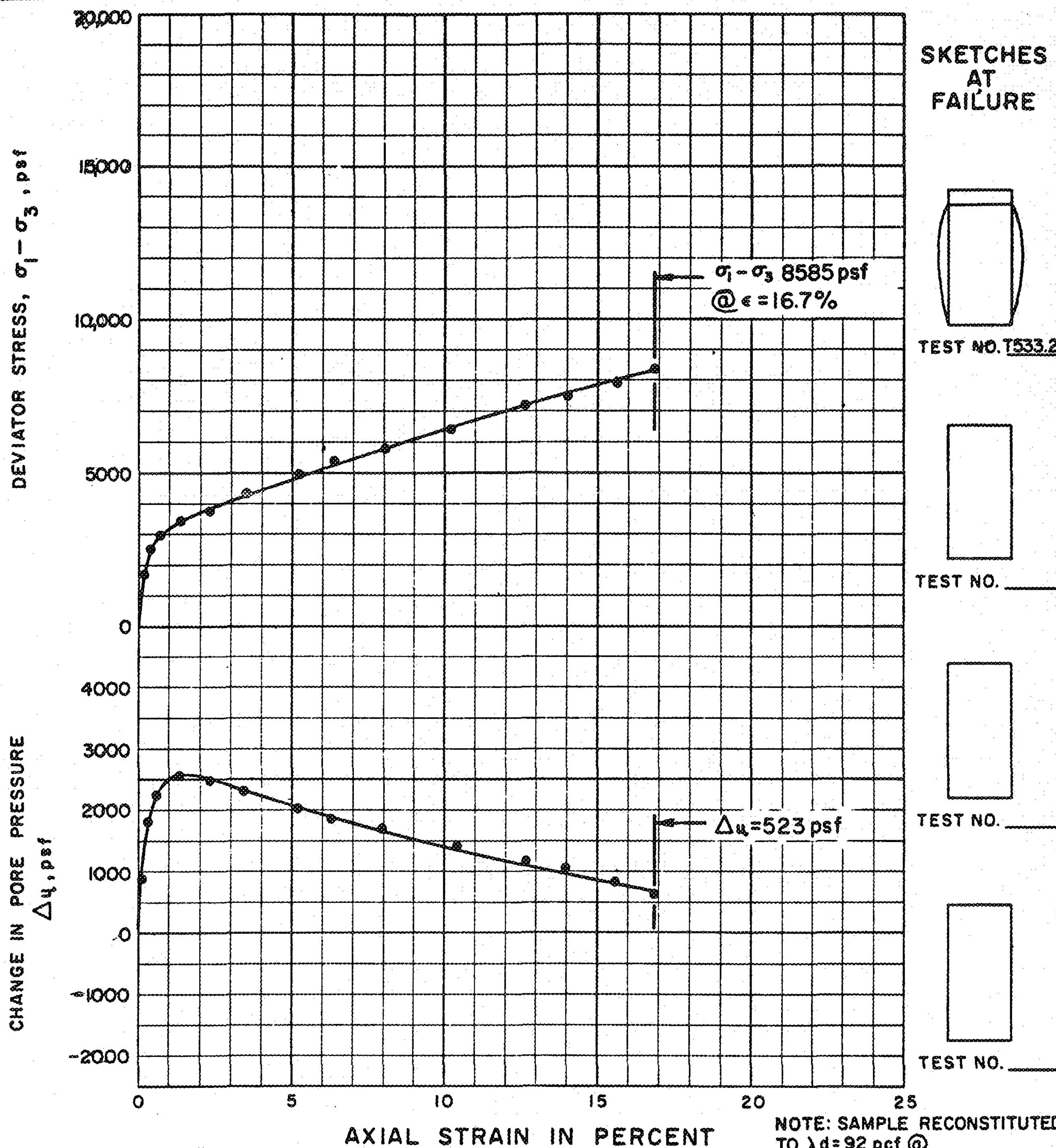
MOHR STRENGTH ENVELOPE

TRIAXIAL COMPRESSION TESTS

BORING NO. SM1 TEST SERIES NO. 53

SAMPLE T1

DEPTH 26.3'-27.7' DATE DEC. 1980



NOTE: SAMPLE RECONSTITUTED TO λd = 92 pcf @ ω = 6.7%

TEST NO./SYMBOL	INITIAL CONDITIONS: INITIAL WATER CONTENT, %	INITIAL CONDITIONS: INITIAL DRY UNIT WEIGHT, pcf	INITIAL CONDITIONS: SAMPLE HEIGHT & DIAMETER, in.	CONDITIONS BEFORE SHEAR: INITIAL STRESSES $\bar{\sigma}_1 = \bar{\sigma}_3$, psf	CONDITIONS BEFORE SHEAR: FINAL BACK PRESSURE, psf	CONDITIONS BEFORE SHEAR: VOLUMETRIC STRAIN, %	CONDITIONS BEFORE SHEAR: PORE PRESSURE RESPONSE, %	FINAL CONDITIONS: FINAL WATER CONTENT, %	FINAL CONDITIONS: FINAL DRY UNIT WEIGHT, pcf	RATE OF STRAIN, PERCENT PER MINUTE
T533.2 ⊙	6.7	92.2	6.000 / 2.900	4320	8640	1.51	95	31.5	93.6	—

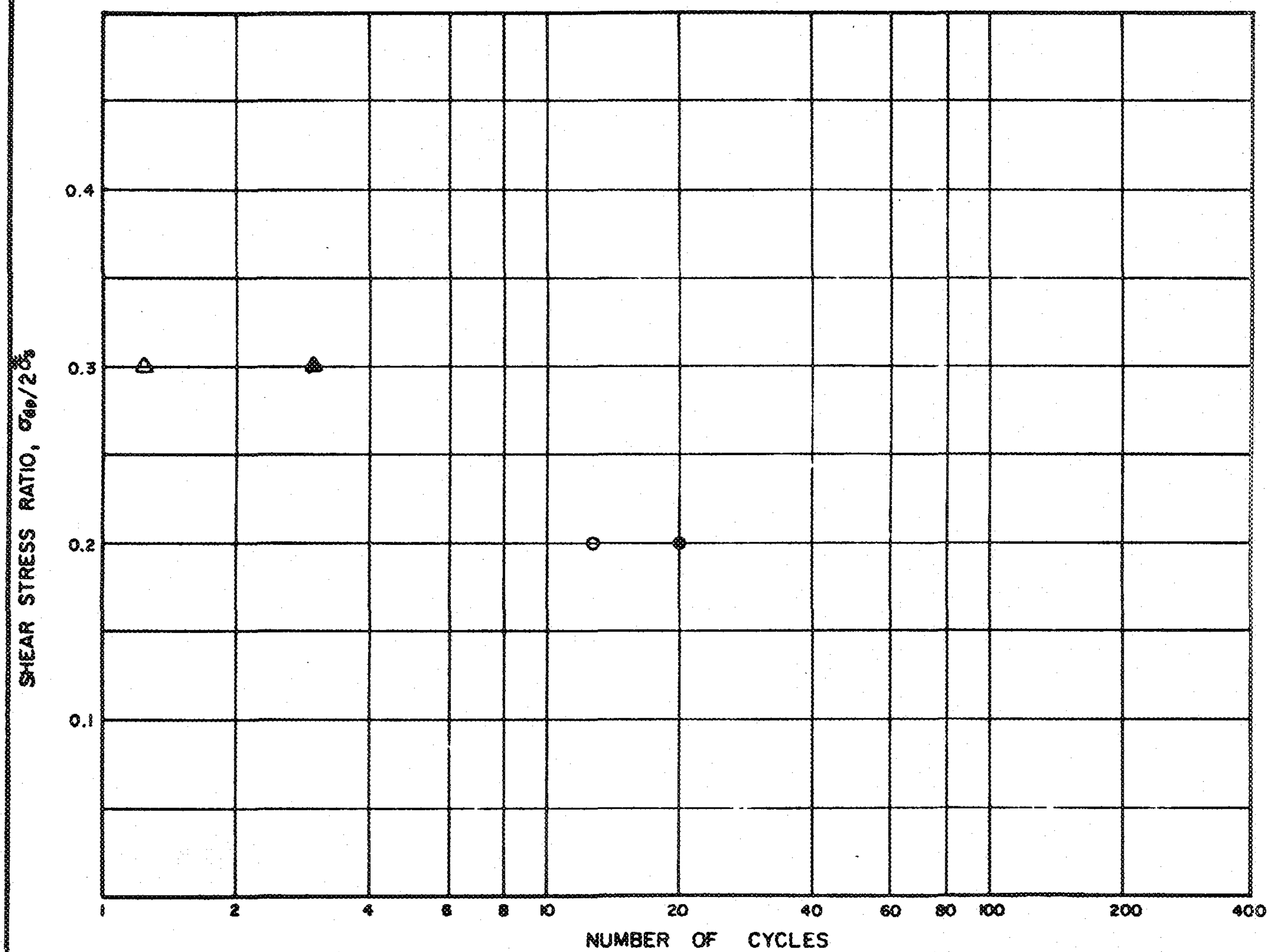
SOIL DESCRIPTION: GREY SILT AND FINE SAND (ML.)

LIQUID LIMIT ____ % PLASTIC LIMIT ____ % SPECIFIC GRAVITY ____

SURRY, MOUNTAIN DAM
SURRY, N.H.
TRIAXIAL COMPRESSION TESTS

BORING NO. SMI
SAMPLE TI
DEPTH 26.3'-27.7'

TEST SERIES NO. 53
DATE DEC. 1980



ϵ_{pp} = DOUBLE APPLITUDE STRAIN

σ_{dp} = CYCLIC DEVIATOR STRESS $(\sigma_1 - \sigma_3)$

O, △ = NUMBER OF CYCLES TO ϵ_{pp} = 5%

⊗, ▲ = NUMBER OF CYCLES TO ϵ_{pp} = 10%

TEST NO. / SYMBOL			53.1 / ⊗	53.2 / ▲			
INITIAL CONDITIONS	WATER CONTENT		27.9%	29.3%	%	%	%
	DRY DENSITY pcf		93.6	91.2			
	RELATIVE DENSITY %		-	-			
	SAMPLE DIAMETER in.		2.830	2.830			
	SAMPLE HEIGHT in.		5.720	5.640			
CONDITIONS BEFORE LOAD APPLICATION	FINAL BACK PRESSURE psf	u_b	7200	7200			
	INITIAL EFFECTIVE STRESS psf	$\bar{\sigma}_{3c}$	5760	5760			
	VOLUMETRIC STRAIN		2.4%	2.7%	%	%	%
	PORE PRESSURE RESPONSE		97	96			
FINAL CONDITIONS	WATER CONTENT	w_f	26.8%	28.5%	%	%	%
	SKETCH OF SAMPLE AT END OF TEST						
LOAD APPLICATION RATE cps			1	1			

BORING No. SM1 SAMPLE No. T1

DEPTH 26.3' - 27.7'

SOIL DESCRIPTION GREY SILT, AND FINE SAND (ML)

ATTERBERG LIMITS

LIQUID LIMIT ____ %, PLASTIC LIMIT ____ %

SURRY MOUNTAIN DAM
SURRY, N.H.

SUMMARY PLOT

SHEAR STRESS RATIO vs CYCLES

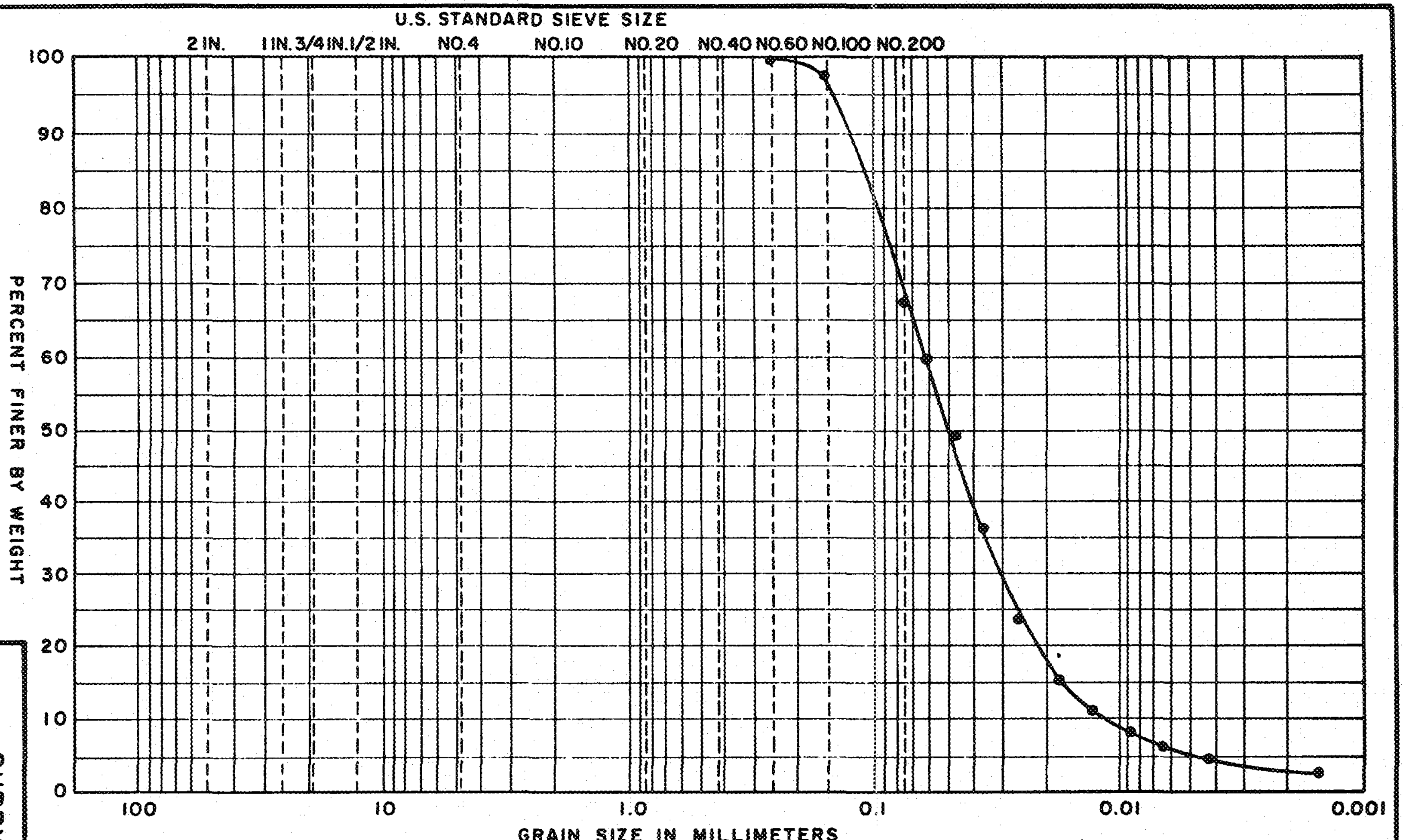
DATE DEC. 1980

SURRY MOUNTAIN DAM
SURRY, N.H.

GRADATION TESTS

BORING NO. SM1
SAMPLE T1
DEPTH 27.0'-27.2'

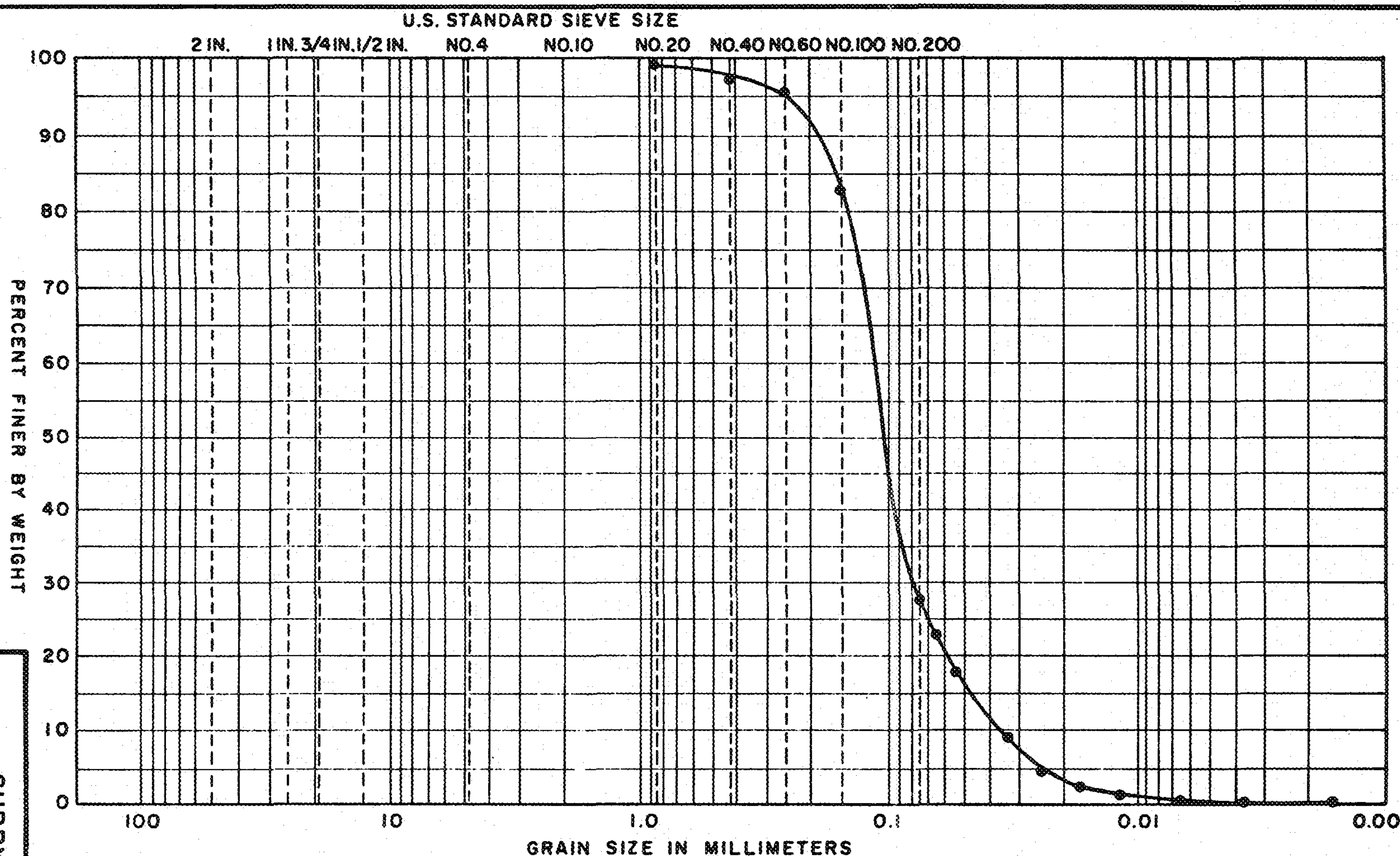
TEST SERIES NO. 53
DATE DEC. 1980



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

UNIFIED SOIL CLASSIFICATION SYSTEM

TEST NO.	SYM.	MATERIAL SOURCE	REMARKS
S/H 53.1		BORING SM1 SAMPLE T1 DEPTH 27.0'-27.2'	GREY SILT AND FINE SAND (ML)



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

UNIFIED SOIL CLASSIFICATION SYSTEM

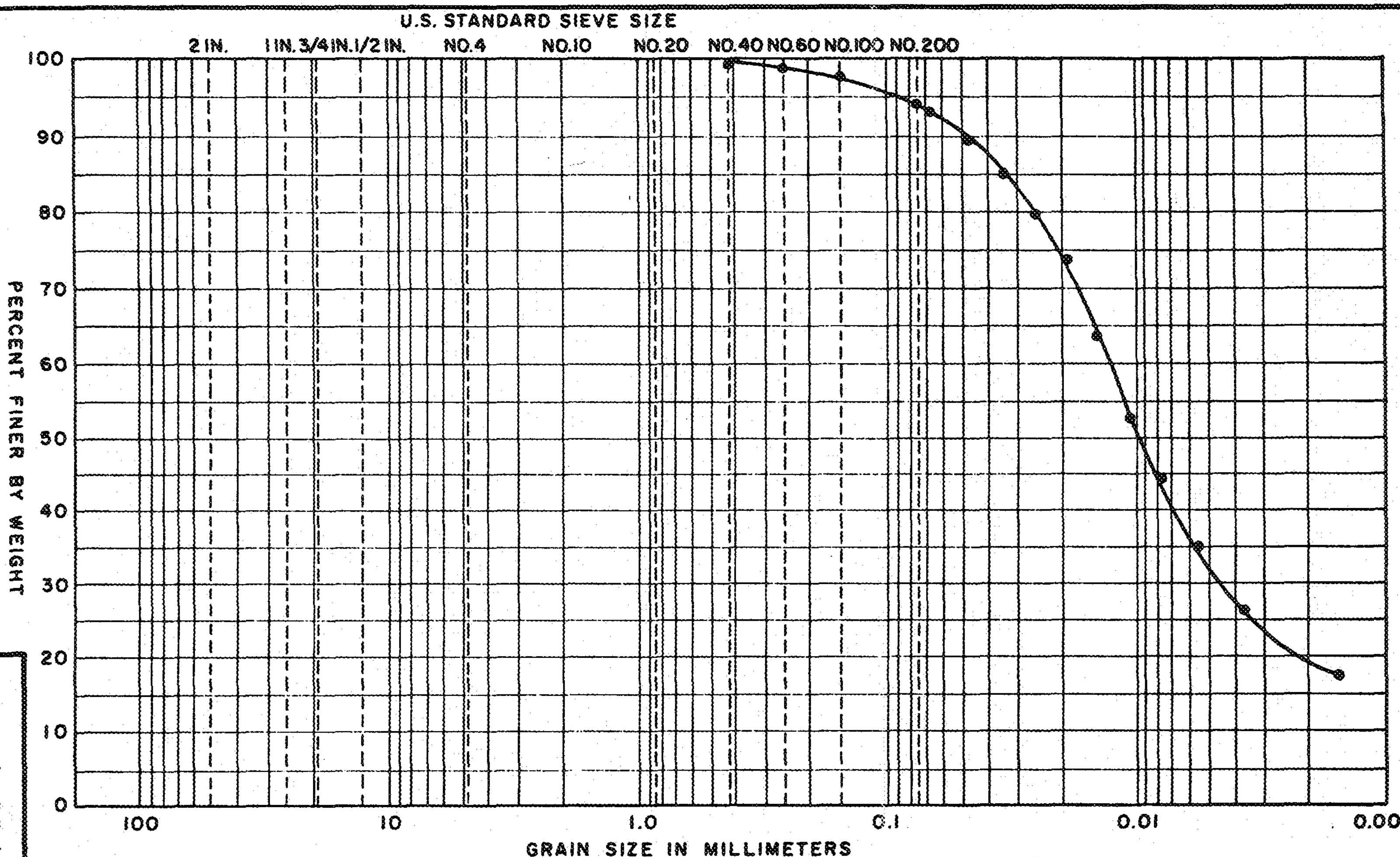
TEST NO.	SYM.	MATERIAL SOURCE	REMARKS
S/H 54.1		BORING SM1 SAMPLE T2 DEPTH 35.9'-36.4'	GREY UNIFORM FINE SAND, SOME SILT (SM)

SURRY MOUNTAIN DAM
SURRY, N.H.

GRADATION TESTS

BORING NO. SM1
SAMPLE T2
DEPTH 35.9'-36.4'

TEST SERIES
NO. 54
DATE DEC.1980



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

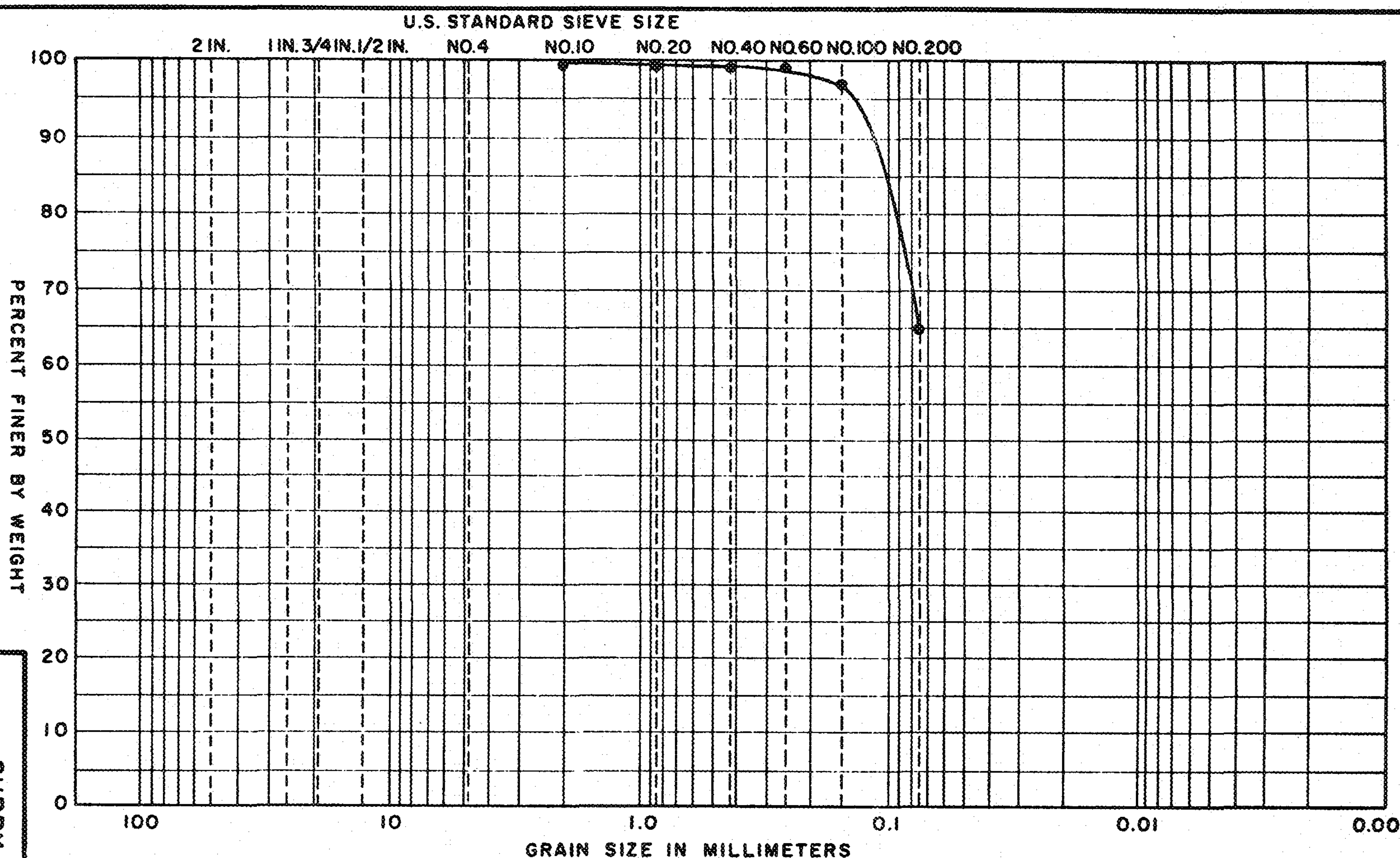
UNIFIED SOIL CLASSIFICATION SYSTEM

TEST NO.	SYM.	MATERIAL SOURCE	REMARKS
S/H 36.1		BORING SM1 SAMPLE S4 DEPTH 18.5'-20.0'	GREY SILT & CLAY OF LOW TO MEDIUM PLASTICITY, TRACE FINE SAND (CL)

SURRY MOUNTAIN DAM
SURRY, N.H.

GRADATION TESTS

BORING NO. SM1 TEST SERIES
SAMPLE S4 NO. 36
DEPTH 18.5'-20.0' DATE NOV. 1980



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

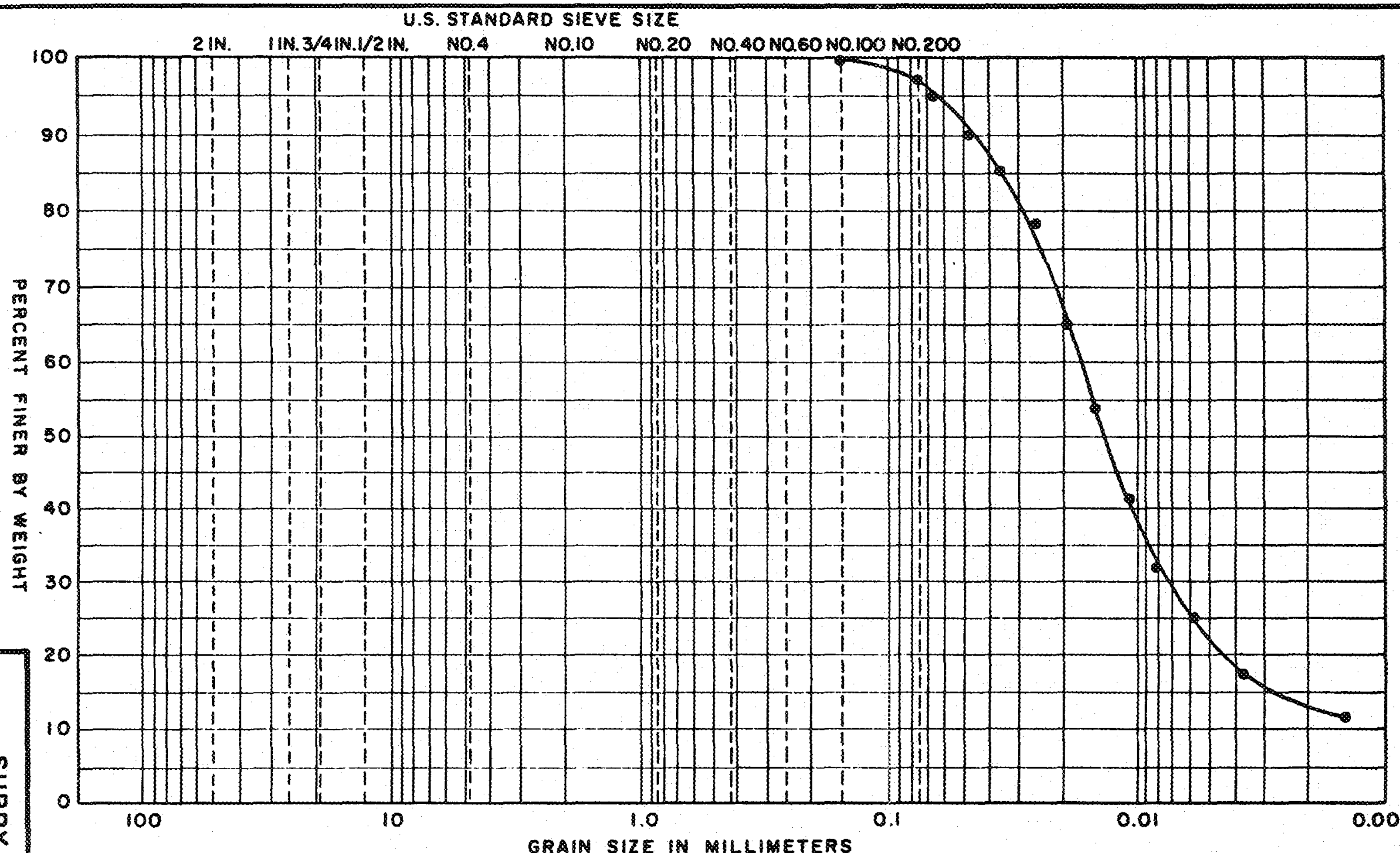
UNIFIED SOIL CLASSIFICATION SYSTEM

TEST NO.	SYM.	MATERIAL SOURCE	REMARKS
S38.1		BORING SM1 SAMPLE S7 DEPTH 48.0'-49.5'	GREY SILT, AND FINE SAND (ML)

SURRY MOUNTAIN DAM
SURRY, N.H.

GRADATION TESTS

BORING NO. SM1 TEST SERIES
SAMPLE S7 NO. 38
DEPTH 48.0'-49.5' DATE NOV.1980



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

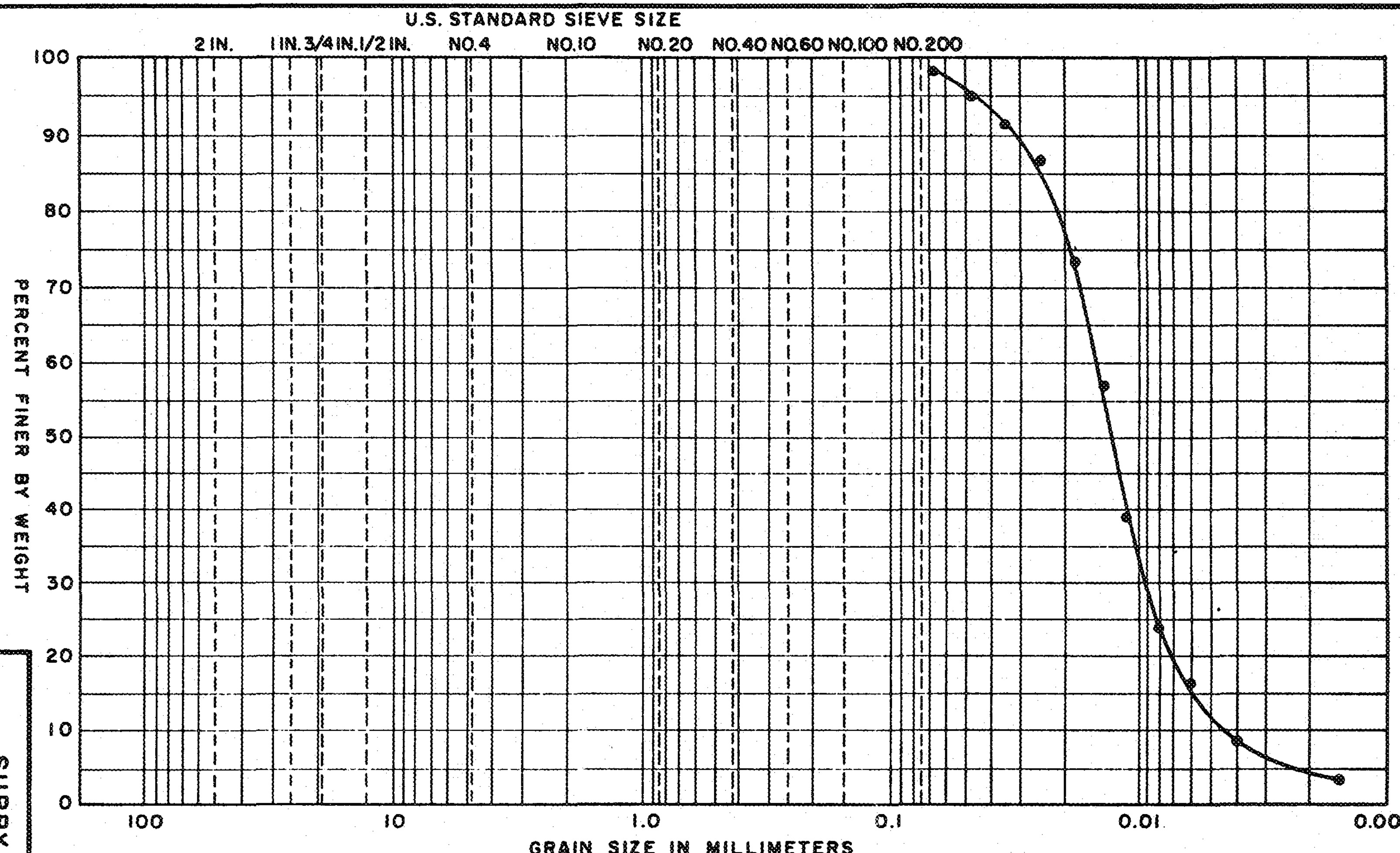
UNIFIED SOIL CLASSIFICATION SYSTEM

TEST NO.	SYM.	MATERIAL SOURCE	REMARKS
S/H 42.1		BORING SM2 SAMPLE S5 DEPTH 24.5'-26.0'	GREY CLAYEY SILT OF SLIGHT PLASTICITY, TRACE FINE SAND (ML)

SURRY MOUNTAIN DAM
SURRY, N.H.

GRADATION TESTS

BORING NO. SM2 TEST SERIES NO. 42
SAMPLE S5
DEPTH 24.5'-26.0' DATE NOV.1980



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

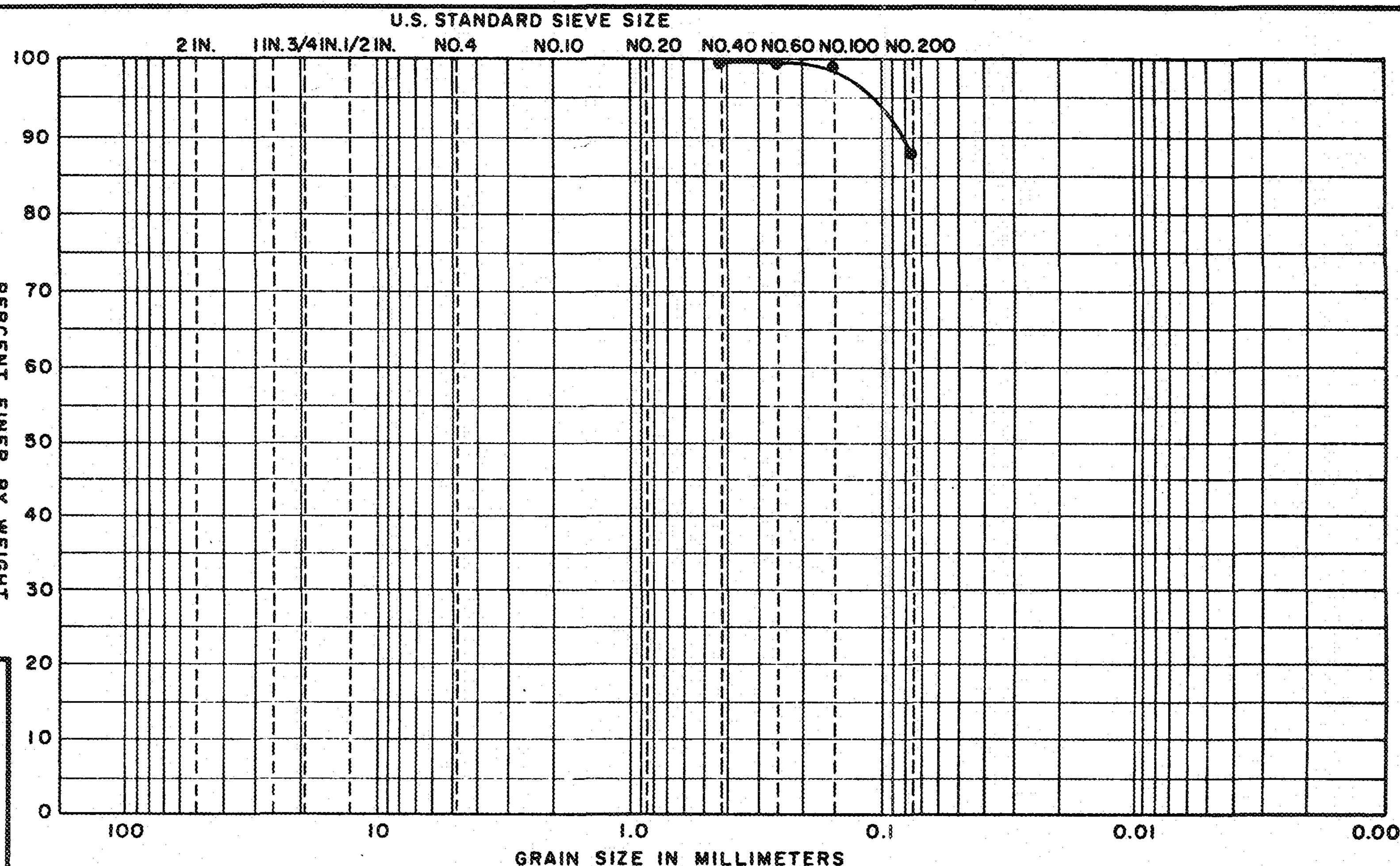
UNIFIED SOIL CLASSIFICATION SYSTEM

TEST NO.	SYM.	MATERIAL SOURCE	REMARKS
H43.1		BORING SM2 SAMPLE S7 DEPTH 34.5'-36.0'	GREY CLAYEY SILT OF SLIGHT PLASTICITY (ML)

SURRY MOUNTAIN DAM
SURRY, N.H.

GRADATION TESTS

BORING NO. SM2 TEST SERIES
SAMPLE S7 NO. 43
DEPTH 34.5'-36.0' DATE NOV.1980



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

UNIFIED SOIL CLASSIFICATION SYSTEM

TEST NO.	SYM.	MATERIAL SOURCE	REMARKS
S44.1		BORING SM2 SAMPLE S9 DEPTH 44.5'-46.0'	GREY SILT, LITTLE FINE SAND (ML)

SURRY MOUNTAIN DAM
SURRY, N.H.

GRADATION TESTS

BORING NO. SM2 TEST SERIES NO. 44
SAMPLE S9
DEPTH 44.5'-46.0' DATE NOV.1980

EXHIBIT D

EXHIBIT D

PSEUDO-STATIC EARTHQUAKE STABILITY ANALYSES*

*The material in this exhibit was obtained from report entitled "Earthquake Design and Analyses for Corps of Engineers Dams, Stability Analyses by the Coefficient Method, Completed New England Division Dams", August 1980.

SURREY MOUNTAIN DAM

The pseudo-static analysis of Surrey Mountain Dam is based on the information contained in the following documents:

> <u>Surry Mountain Dam - Analysis of Design</u>, Corps of Engineers, U. S. Army, U. S. Engineer Office, Providence, Rhode Island, Revised July 1939.

<u>Cross Section Analyzed</u>

The cross section at Sta 11+00 (Plate 42 in the Design Memorandum) was analyzed for stability of the upstream and downstream slopes. This is the maximum height section of the dam.

<u>Embankment and Foundation Materials</u>

The shear strengths and unit weights adopted for the pseudo-static analyses were based on the data presented in the Design Report. The shear strength data in the Design Report is discussed below for each material, numbered in accordance with the material numbers presented in Figs. A227 to A234. Comments are made when the shear strength adopted for the pseudo-static analysis was not based on data given in the Design Report.

1. Dumped Rock. No strength for the dumped rock was given in the Design Report. The strength used for the pseudo-static analysis was selected on the basis of experience with similar materials.

2. Pervious Fill. The drained shear strength data in the Design Report are based on the results of direct shear tests.

3. Random Impervious. The shear strength parameters presented in the Design Report were based on the results of direct shear tests.

4. Select Impervious. The shear strength data presented in the Design Report were based on the results of direct shear tests.

5. Foundation Sand. The shear strength data presented in the Design Report correspond to direct shear tests.

6. Foundation Silt. Shear strength tests on the silt presented in the Design Report incorporated varying rates of shear but they were probably all drained tests. The shear strength value selected in the Design Report corresponded to the lowest range of values measured.

Selection of Phreatic Surface

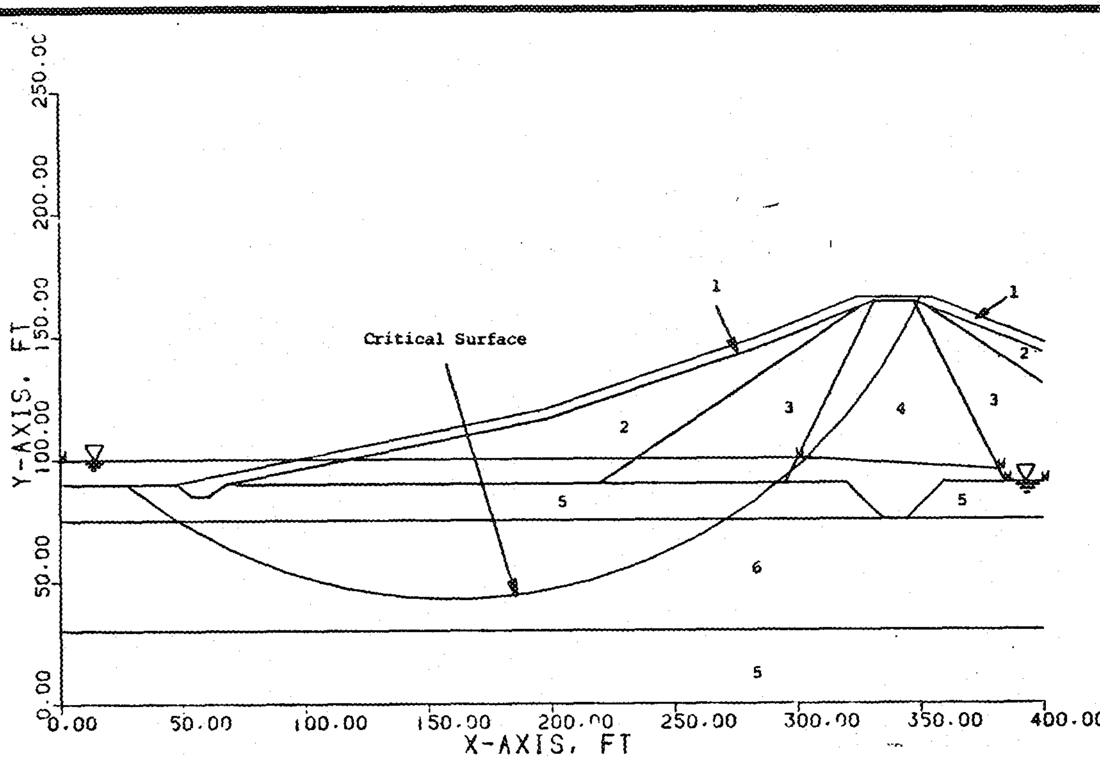
Approximate phreatic surfaces for various pool elevations were constructed using the analytical solutions for location of the seepage exit point presented by R. Lo (1969). The horizontal permeability of the impervious zone was assumed to be nine times the vertical permeability. Based on the permeability values presented on p. 39 of the Design Memorandum, the permeability of the Random Impervious material was taken to be 100 times that of the Select Impervious material and, therefore, the Random Material downstream of the Select Impervious core was assumed to be completely drained. The phreatic surfaces used for the pseudo-static analysis are somewhat different than the phreatic surface shown on Plate 46 in the Design Memorandum.

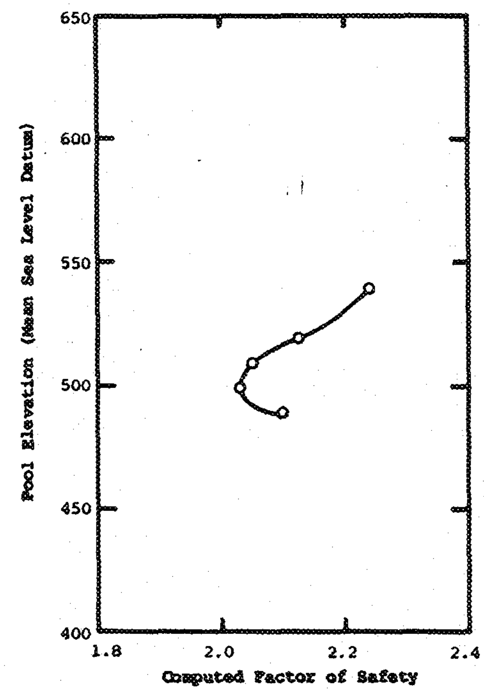
Potential Failure Surfaces Analyzed

Potential failure surfaces analyzed for the upstream and downstream slopes were circular surfaces extending from the crest and upper portion of the slope behind the crest to the break in slope at El 520, the lower portion of the slope in the vicinity of the toe, and the ground surface beyond the toe.

Comments on Results

The stability analyses presented in the Design Memorandum consist of rapid drawdown and "complete capillary saturation" analyses using stability charts based on the Friction-Circle Method and a sliding-wedge analysis for a deep failure through the foundation silt (Material No. 6 in Figs. A227 to A234) which is apparently an analysis for the condition at the end of construction. A meaningful comparison cannot be made between the analyses presented in the Design Memorandum and the static analyses performed for the present investigation.

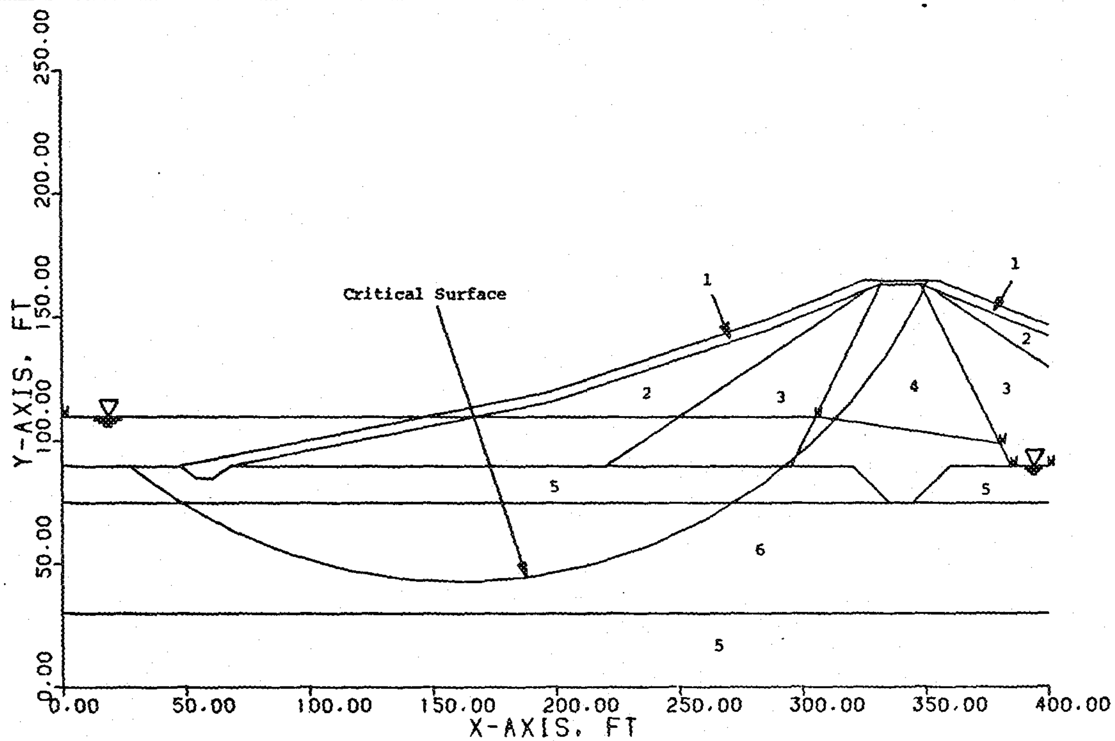


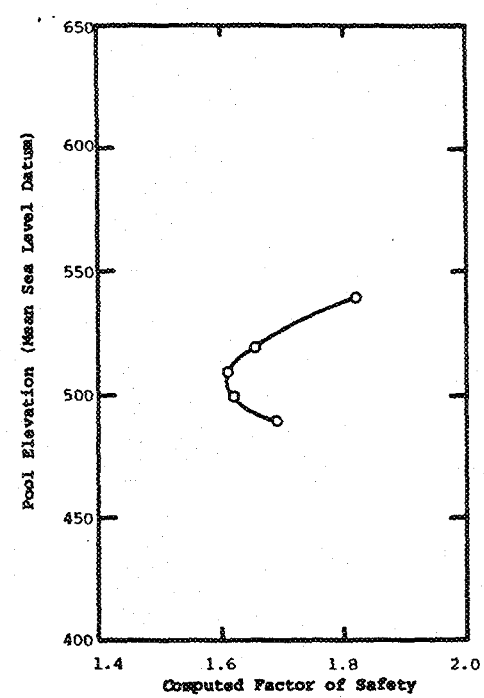


Earthquake Acceleration: 0
Minimum Computed Factor of Safety: 2.03
Critical Pool Elevation: 500

Material	Unit Weights		Shear Strength					
	γ_m	γ_{sat}	S		R		Adopted	
	pcf	pcf	c ksf	ϕ deg	c ksf	ϕ deg	c ksf	ϕ deg
1	110	132	0	40	-	-	0	40
2	130	136	0	40	-	-	0	40
3	135	139	0	37	-	-	0	37
4	140	142	0	34	-	-	0	34
5	132	132	0	37	-	-	0	37
6	125	125	0	25	-	-	0	25

U.S. Corps of Engineers Waltham, Massachusetts	Pseudo-Static Analysis New England Dams	SURREY MOUNTAIN DAM Upstream Slope Sta 11+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 80678	July 23, 1980 Fig. A227

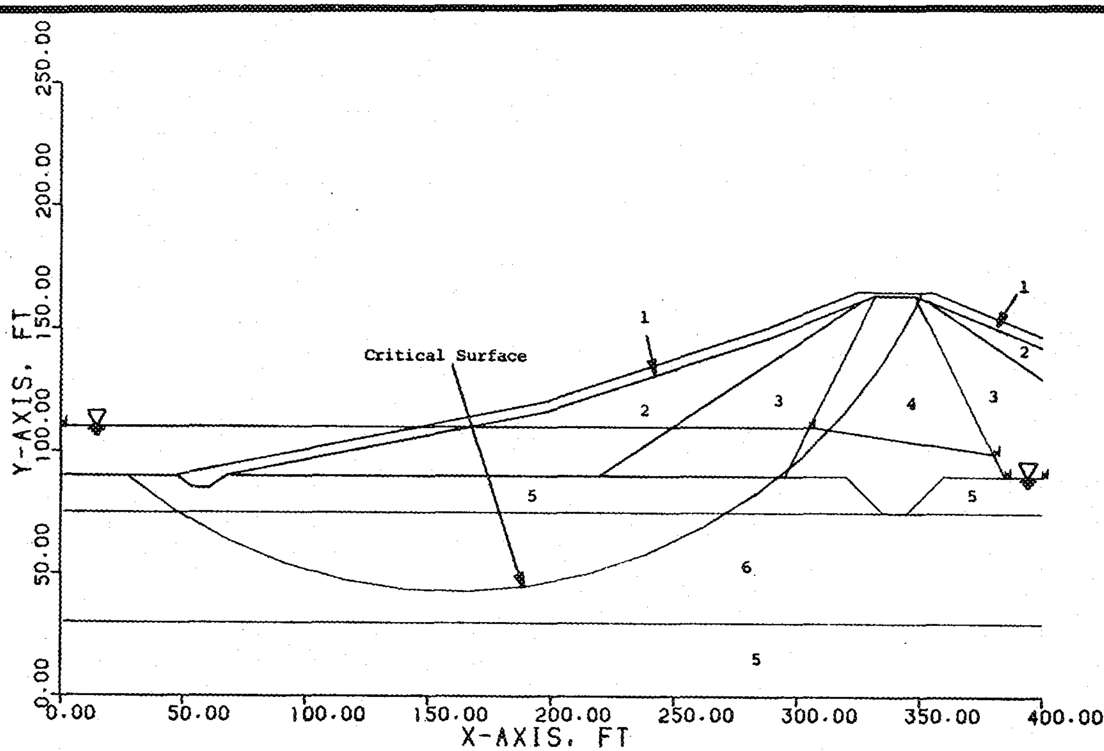


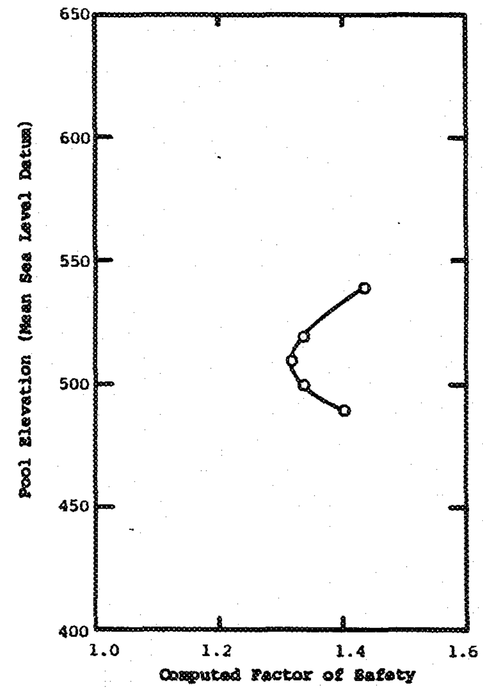


Material	Unit Weights		Shear Strength					
	γ_m	γ_{sat}	S		R		Adopted	
	pcf	pcf	c ksf	ϕ deg	c ksf	ϕ deg	c ksf	ϕ deg
1	110	132	0	40	-	-	0	40
2	130	136	0	40	-	-	0	40
3	135	139	0	37	-	-	0	37
4	140	142	0	34	-	-	0	34
5	132	132	0	37	-	-	0	37
6	125	125	0	25	-	-	0	25

Earthquake Acceleration: 0.05g
Minimum Computed Factor of Safety: 1.61
Critical Pool Elevation: 510

U.S. Corps of Engineers Waltham, Massachusetts	Pseudo-Static Analysis New England Dams	SURREY MOUNTAIN DAM Upstream Slope Sta 11+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 80678	July 23, 1980 Fig. A228





Earthquake Acceleration: 0.10g

Minimum Computed Factor of Safety: 1.32

Critical Pool Elevation: 510

Material	Unit Weights		Shear Strength					
			S		R		Adopted	
	γ_m pcf	γ_{sat} pcf	c ksf	ϕ deg	c ksf	ϕ deg	c ksf	ϕ deg
1	110	132	0	40	-	-	0	40
2	130	136	0	40	-	-	0	40
3	135	139	0	37	-	-	0	37
4	140	142	0	34	-	-	0	34
5	132	132	0	37	-	-	0	37
6	125	125	0	25	-	-	0	25

U.S. Corps of Engineers
Waltham, Massachusetts

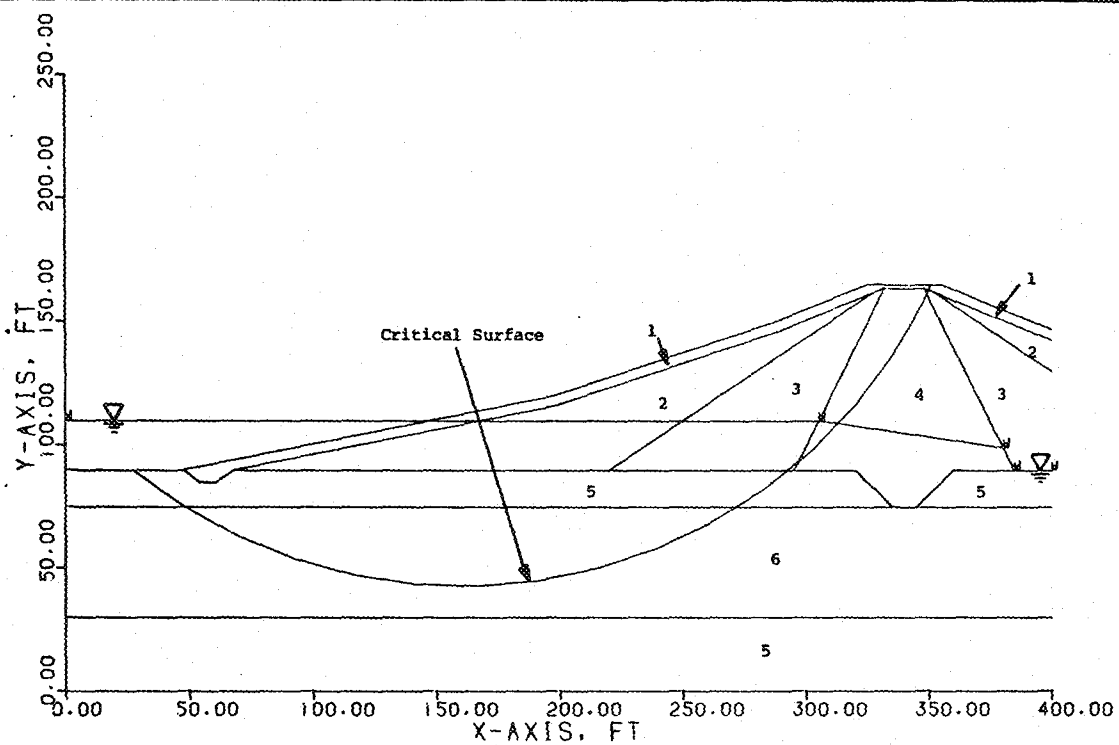
GEOTECHNICAL ENGINEERS INC
WINCHESTER • MASSACHUSETTS

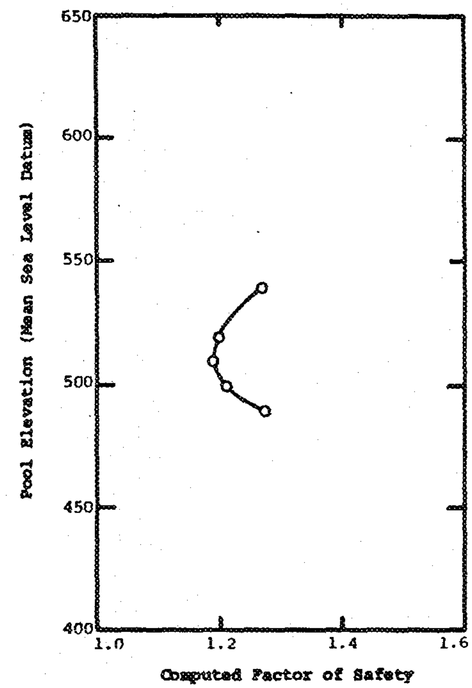
Pseudo-Static Analysis
New England Dams

Project 80678

SURREY MOUNTAIN DAM
Upstream Slope
Sta 11+00

July 23, 1980 Fig. A229





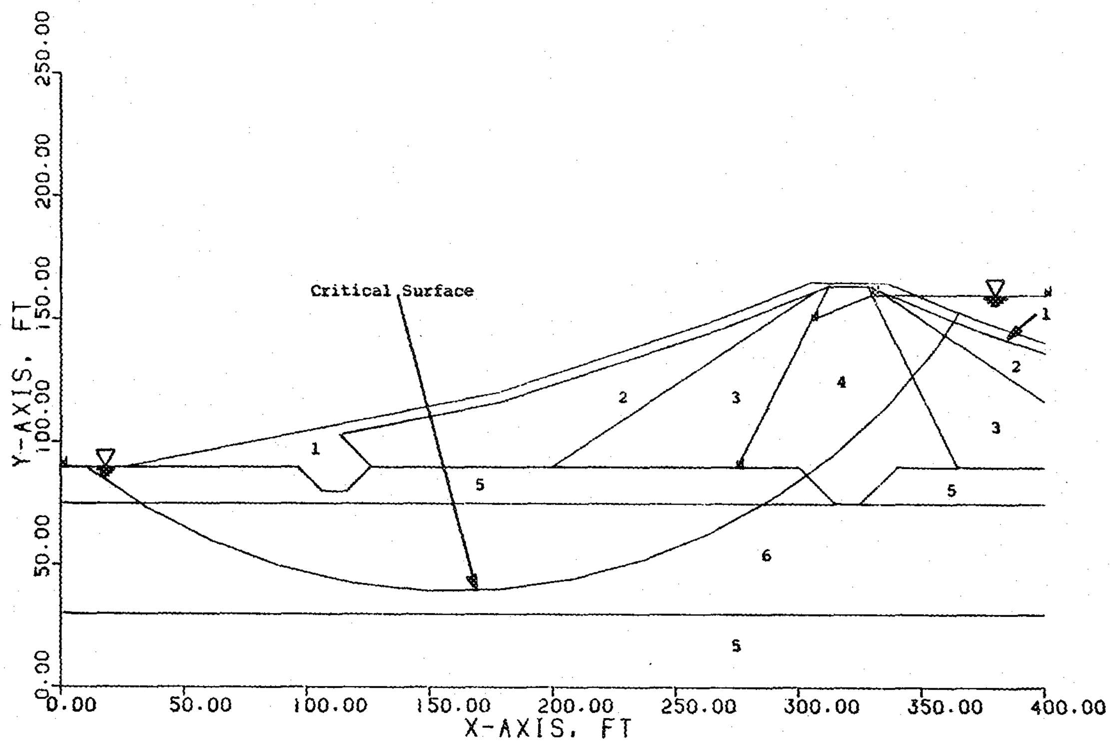
Earthquake Acceleration: 0.13g

Minimum Computed Factor of Safety: 1.19

Critical Pool Elevation: 510

Material	Unit Weights		Shear Strength					
			S		R		Adopted	
	γ_m pcf	γ_{sat} pcf	c ksf	φ deg	c ksf	φ deg	c ksf	φ deg
1	110	132	0	40	-	-	0	40
2	130	136	0	40	-	-	0	40
3	135	139	0	37	-	-	0	37
4	140	142	0	34	-	-	0	34
5	132	132	0	37	-	-	0	37
6	125	125	0	25	-	-	0	25

U.S. Corps of Engineers Waltham, Massachusetts	Pseudo-Static Analysis New England Dams	SURREY MOUNTAIN DAM Upstream Slope Sta 11+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 80678	July 23, 1980 Fig. A230



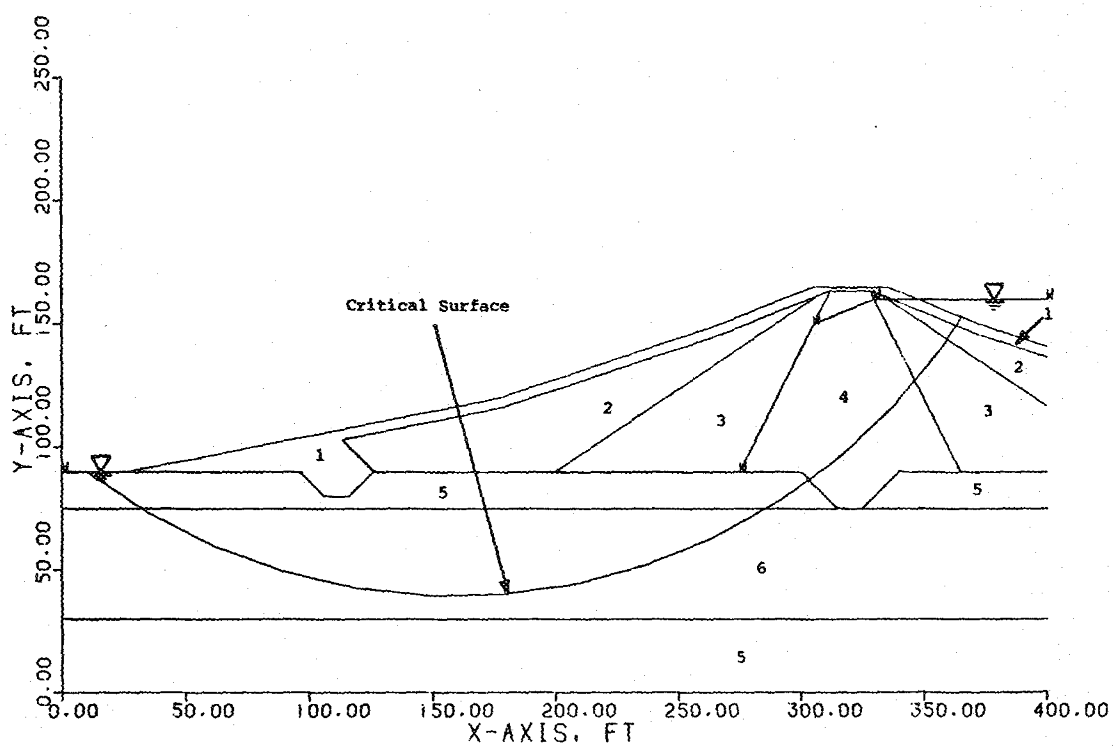
Material	Unit Weights		Shear Strength					
	γ_m	γ_{sat}	S		R		Adopted	
	pcf	pcf	c ksf	ϕ deg	c ksf	ϕ deg	c ksf	ϕ deg
1	110	132	0	40	-	-	0	40
2	130	136	0	40	-	-	0	40
3	135	139	0	37	-	-	0	37
4	140	142	0	34	-	-	0	34
5	132	132	0	37	-	-	0	37
6	125	125	0	25	-	-	0	25

Earthquake Acceleration: 0

Minimum Computed Factor of Safety: 1.86

Maximum Pool Elevation: 560

U.S. Corps of Engineers Waltham, Massachusetts	Pseudo-Static Analysis New England Dams	SURREY MOUNTAIN DAM Downstream Slope Sta 11+00
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	Project 80678	July 23, 1980 Fig. A231



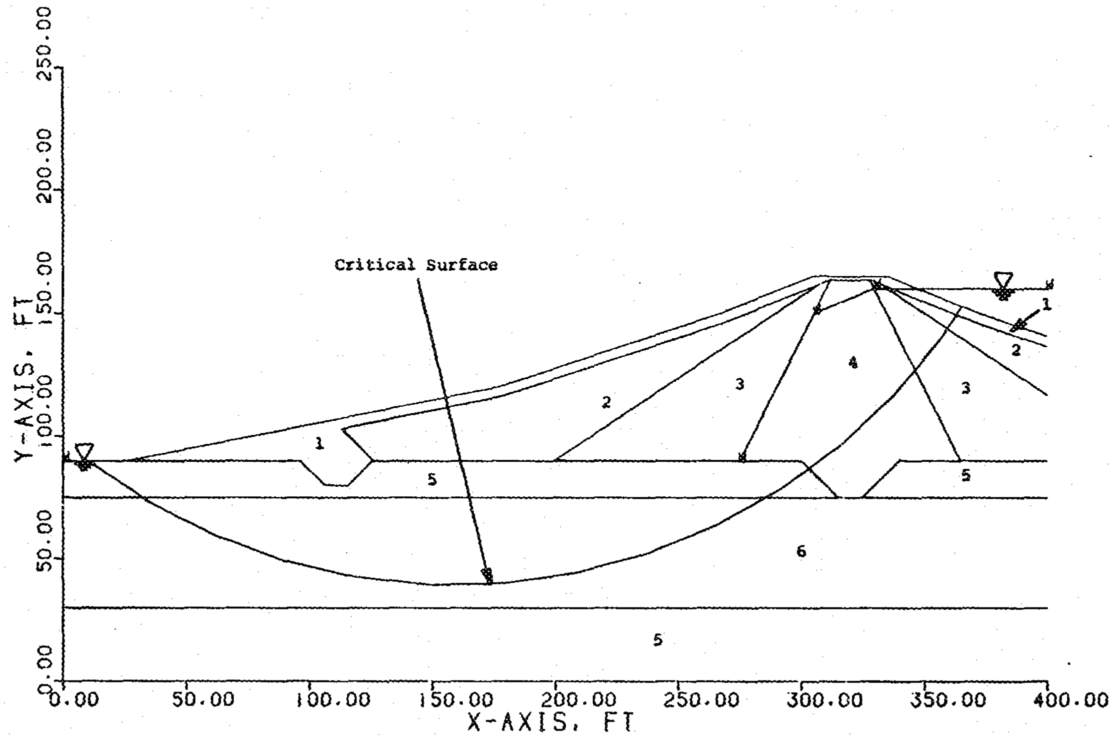
Material	Unit Weights		Shear Strength					
	γ_m	γ_{sat}	S		R		Adopted	
	pcf	pcf	c ksf	ϕ deg	c ksf	ϕ deg	c ksf	ϕ deg
1	110	132	0	40	-	-	0	40
2	130	136	0	40	-	-	0	40
3	135	139	0	37	-	-	0	37
4	140	132	0	34	-	-	0	34
5	132	132	0	37	-	-	0	37
6	125	125	0	25	-	-	0	25

Earthquake Acceleration: 0.05g

Minimum Computed Factor of Safety: 1.50

Maximum Pool Elevation: 560

U.S. Corps of Engineers Waltham, Massachusetts	Pseudo-Static Analysis New England Dams	SURREY MOUNTAIN DAM Downstream Slope Sta 11+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 80678	July 23, 1980 Fig. A232



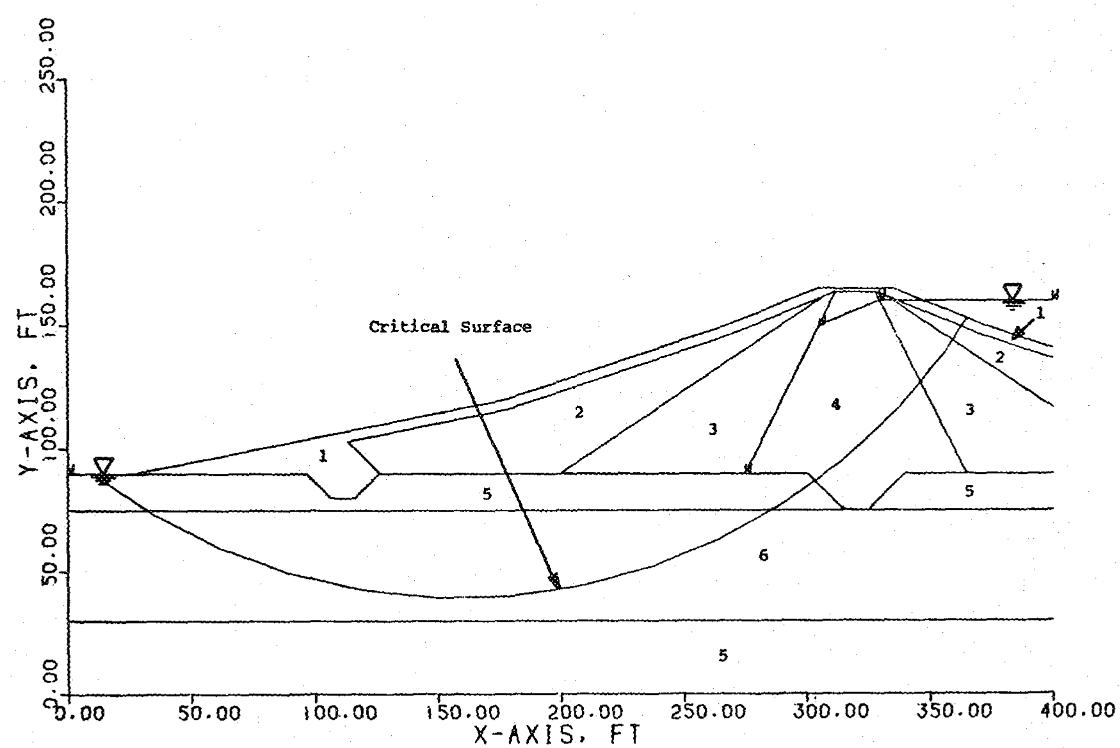
Material	Unit Weights		Shear Strength					
	γ_m	γ_{sat}	S		R		Adopted	
	pcf	pcf	c ksf	ϕ deg	c ksf	ϕ deg	c ksf	ϕ deg
1	110	132	0	40	-	-	0	40
2	130	136	0	40	-	-	0	40
3	135	139	0	37	-	-	0	37
4	140	142	0	34	-	-	0	34
5	132	132	0	37	-	-	0	37
6	125	125	0	25	-	-	0	25

Earthquake Acceleration: 0.10g

Minimum Computed Factor of Safety: 1.26

Maximum Pool Elevation: 560

U.S. Corps of Engineers Waltham, Massachusetts	Pseudo-Static Analysis New England Dams	SURREY MOUNTAIN DAM Downstream Slope Sta 11+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 80678	July 23, 1980 Fig. A233



Material	Unit Weights γ_m pcf	γ_{sat} pcf	Shear Strength S c ksf	S ϕ deg	R c ksf	R ϕ deg	Adopted c ksf	Adopted ϕ deg
1	110	132	0	40	-	-	0	40
2	130	136	0	40	-	-	0	40
3	135	139	0	37	-	-	0	37
4	140	142	0	34	-	-	0	34
5	132	132	0	37	-	-	0	37
6	125	125	0	25	-	-	0	25

Earthquake Acceleration: 0.13g

Minimum Computed Factor of Safety: 1.15

Maximum Pool Elevation: 560

U.S. Corps of Engineers Waltham, Massachusetts	Pseudo-Static Analysis New England Dams	SURREY MOUNTAIN DAM Downstream Slope Sta 11+00
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 80678	July 23, 1980 Fig. A234